CHAPTER 13

STORM DRAINAGE SYSTEMS

Chapter 13 Storm Drain Systems
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13.1 System Definition

13.1.1 Introduction

Storm drain systems are the collection of pipes and channels that serve to convey storm drain flows for their individual collection points to their discharge point. The storm drain system should be designed to operate without inhibiting the capture capacity of the inlets. For ordinary conditions, storm drains are sized on the assumption that they will flow full or practically full under the design discharge but will not flow under pressure head. Using the inlet locations determined in Chapter 12, the design of the system first involves the determination of the location and alignment of the pipes, the tailwater condition at the outfall, and then sizing the pipes to satisfy the prescribed hydraulic grade line (HGL) and energy grade line (EGL) criteria. The Manning's formula is used for capacity calculations.

13.1.2 Design Approach

The design of the storm drain system integrates the location of inlets, the connecting pipes and the outfall location including the tailwater elevation. The initial design steps begin with evaluating the location of the connecting pipes and the outfall in both plan and profile. A preliminary storm drain profile is laid out from the outfall to the inlets upstream. Consideration must be given to the invert elevations on the inlets, the most upstream inlet might not be the one that controls the slope of the trunk line. The next logical step is the computation of the rate of discharge to be carried by each reach of the storm drain, and the determination of the size and gradient of pipe required to convey this discharge. This is done by starting at the upstream reach, calculating the discharge and sizing the pipe, then proceeding downstream, reach by reach to the point where the storm drain connects with other drains or the outfall.

The design discharge at any point in the storm drain is not the simple addition of the inlet flows of all inlets above that section of storm drain unless the t_c is less than the minimum t_c. It is generally less than this total. The design rate of flow is the sum of the contributing drainage areas and the rainfall intensity for the time of concentration where the additional drainage area occurs. The time of concentration is most influential and as the time of concentration grows larger, the rainfall intensity to be used in the design grows smaller. In some cases, where a relatively large drainage area with a short time of concentration is added to the system, the peak flow may be larger for this subarea using the shorter time even though the entire drainage basin is not contributing. The prudent designer will be alert for unusual conditions and determine which time of concentration controls for each pipe segment. See Section 13.3.6 for a discussion on time of concentration.

At locations where the pipe size is increased (manholes or junction structures), the upstream pipe should be within the end area of the downstream pipe. It is usual to align the crown of pipes for situations with either open channel flow or only slight pressure flow. In summary the following items shall be considered during layout of the storm drain system:

1.) Line up crowns if possible, if not possible overlap the areas.
2.) Use wyes or tees rather than access structures to bring flows into the system. This will bring the connecting pipe at the spring line if the connecting pipe is 0.5D or less of the through pipe.
3.) When discharging into box culverts, bring the connecting pipe at least 18” above the invert.
4.) When possible position the outlet so it is pointed downstream.
5.) If need to do a 90 degree bend in a trunk line, consider 2-45 degree bends offset by 10 pipe diameters.

13.2 Symbols and Definitions

13.2.1 Symbols

To provide consistency within this chapter as well as throughout this manual the symbols in Table 13-1 will be used. These symbols were selected because of their wide use in storm drainage publications.

Table 13-1 Symbols And Definitions

Symbol	Definition	Units
A	Area of cross section	ft^2
A	Watershed area	acres
a	Depth of depression	inches
C	Runoff coefficient or coefficient	-
d	Depth of gutter flow at the curb line	ft
D	Diameter of pipe	ft
H	Head loss	ft
I	Rainfall intensity	in./hr
K	Coefficient	-
L	Length of curb opening inlet	ft.
L	Pipe length	ft.
L	Length of runoff travel	ft.
n	Roughness coefficient in Manning formula	-
Q_i	Intercepted flow	ft^3/sec
Q_T	Total flow	ft^3/sec
R_h	Hydraulic radius	ft
S or S_x	Pavement cross slope	ft/ft
S	Crown slope of pavement	ft/ft
S or S_L	Longitudinal slope of pavement	ft/ft
S_w	Depressed section slope	ft/ft
T	Top width of water surface (spread on pavement)	ft
t_c	Time of concentration	min
V	Velocity of flow	ft/sec
y	Depth of flow in approach gutter	ft

13.2 Symbols and Definitions (continued)

13.2.2 Definitions

Following are discussions of terms that will be used throughout the remainder of this chapter in dealing with different aspects of storm drainage analysis.

Bypass/Flowby (Carry over) -- Occurs at an inlet on grade. It is the flow that is not captured at an inlet on grade, bypasses, and is carried to the next inlet downgrade. Inlets on grade are usually designed to allow a certain amount of flowby, unless located upstream of an area where pedestrians are expected to use the street.

Crown-- The crown, sometimes known as soffit, is the top inside of a pipe.

Culvert-- A culvert is a closed conduit whose purpose is to convey surface water under a roadway, rail-road or other impediment. It may have inlets connected to it.

Flow-- Flow refers to a quantity of water that is flowing.

Hydraulic Grade Line-- The hydraulic grade line is the locus of elevations to which the water would rise in successive piezometer tubes if the tubes were installed along a pipe run (pressure head plus elevation head).

Invert-- The invert is the inside bottom of the pipe.

Lateral Line-- A lateral line, sometimes referred to as a connector, has inlets connected to it but has no other storm drains connected. It is usually tributary to the trunk line.

Pressure Head-- Pressure head is the height of a column of water that would exert a unit pressure equal to the pressure of the water.

Storm Drain-- A storm drain is a closed or open conduit that conveys storm water that has been collected by inlets to an outfall. It generally consists of laterals, connectors, and trunk lines or mains. Culverts connected to the storm drainage system are considered part of the system.

Trunk Line-- A trunk line is the main storm drain line. Lateral lines may be connected in at inlet structures or access holes. A trunk line is sometimes referred to as a "main."

Velocity Head-- Velocity head is a quantity proportional to the kinetic energy of flowing water expressed as a height or head of water, ($V^2/2g$).

13.3 Design Concepts

13.3.1 Hydraulic Capacity

The most widely used formula for determining the hydraulic capacity of storm drains for gravity and pressure flows is the Manning's formula and it is expressed by the following equation:

$$V=\frac{1.486R^{0.67}S^{0.5}}{n} \quad (13.1)$$

Where: V = mean velocity of flow, ft/sec
n = Manning's roughness coefficient
R = hydraulic radius, ft = area of flow divided by the wetted perimeter (A/WP)
S = the slope of the energy grade line, ft/ft

In terms of discharge, the above formula becomes:

$$Q= V*A = \frac{1.486*A*R^{0.67}S^{0.5}}{n} \quad (13.2)$$

Where: Q = rate of flow, ft^3/sec
A = cross sectional area of flow, ft^2

For storm drains flowing full, the above equations become:

$$V= \frac{0.59D^{0.67}S^{0.5}}{n} \quad (13.3)$$

$$Q= \frac{0.46\ D^{2.67}\ S^{0.5}}{n} \quad (13.4)$$

Where: D = diameter of pipe, ft

The nomograph solution of Manning's formula for full flow in circular storm drains is shown on Figure 13-1 and Figure 13-2. Figure 13-3 is provided to assist in the solution of the Manning's equation for part full flow in storm drains.

The slope required for full flow can be determined by rearranging equation 13-4 as

$$S= [Qn/(0.46D^{2.67})]^2 \quad (13.5)$$

Manning's n for commonly used pipe	
Concrete, precast	0.012
Concrete, site cast	0.014
HDPE, smooth interior	0.012

13.3 Design Concepts (continued)

13.3.1 Hydraulic Capacity (continued)

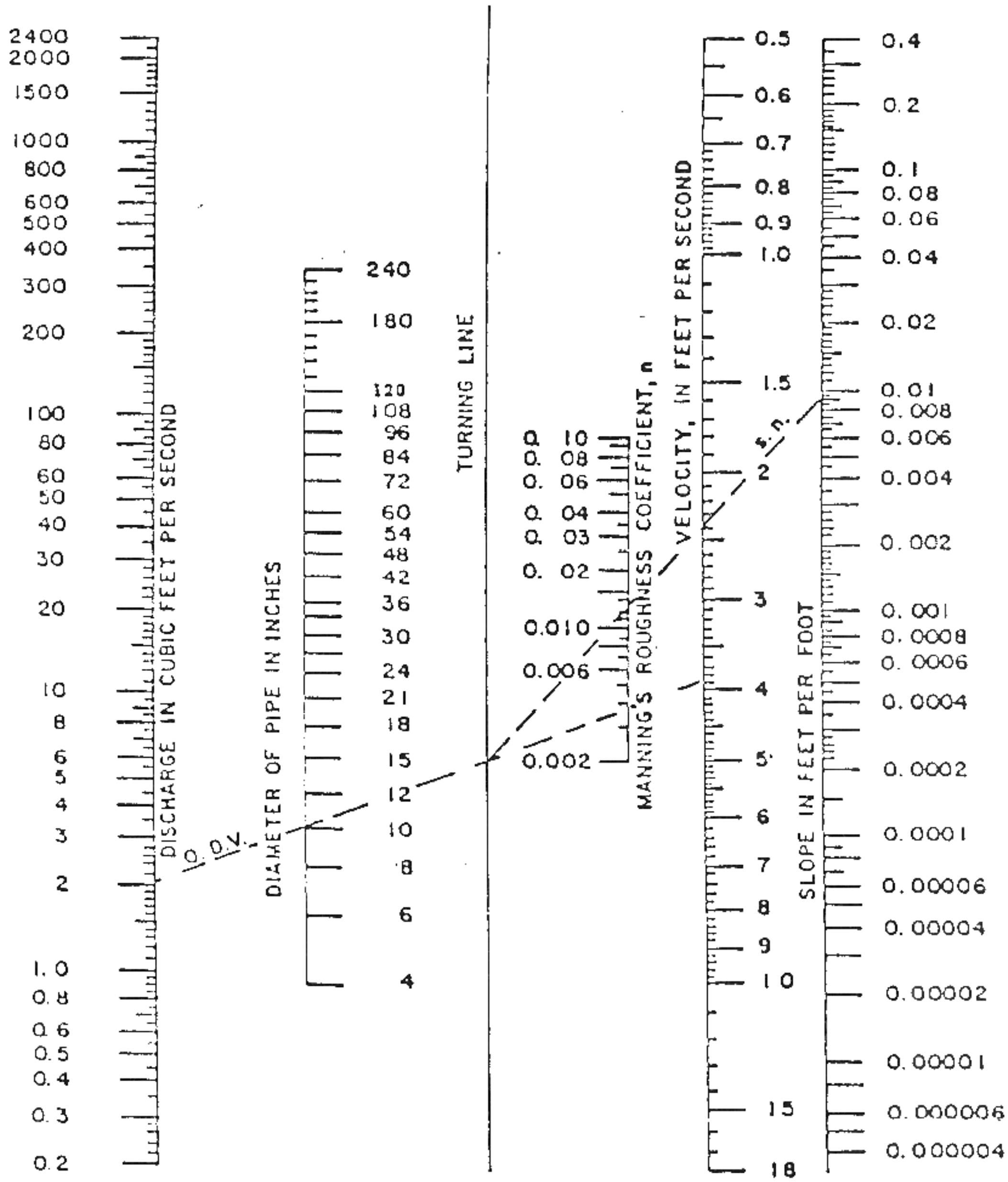


Figure 13-1 Manning's Formula For Flow In Storm Drain

13.3 Design Concepts (continued)

13.3.1 Hydraulic Capacity (continued)

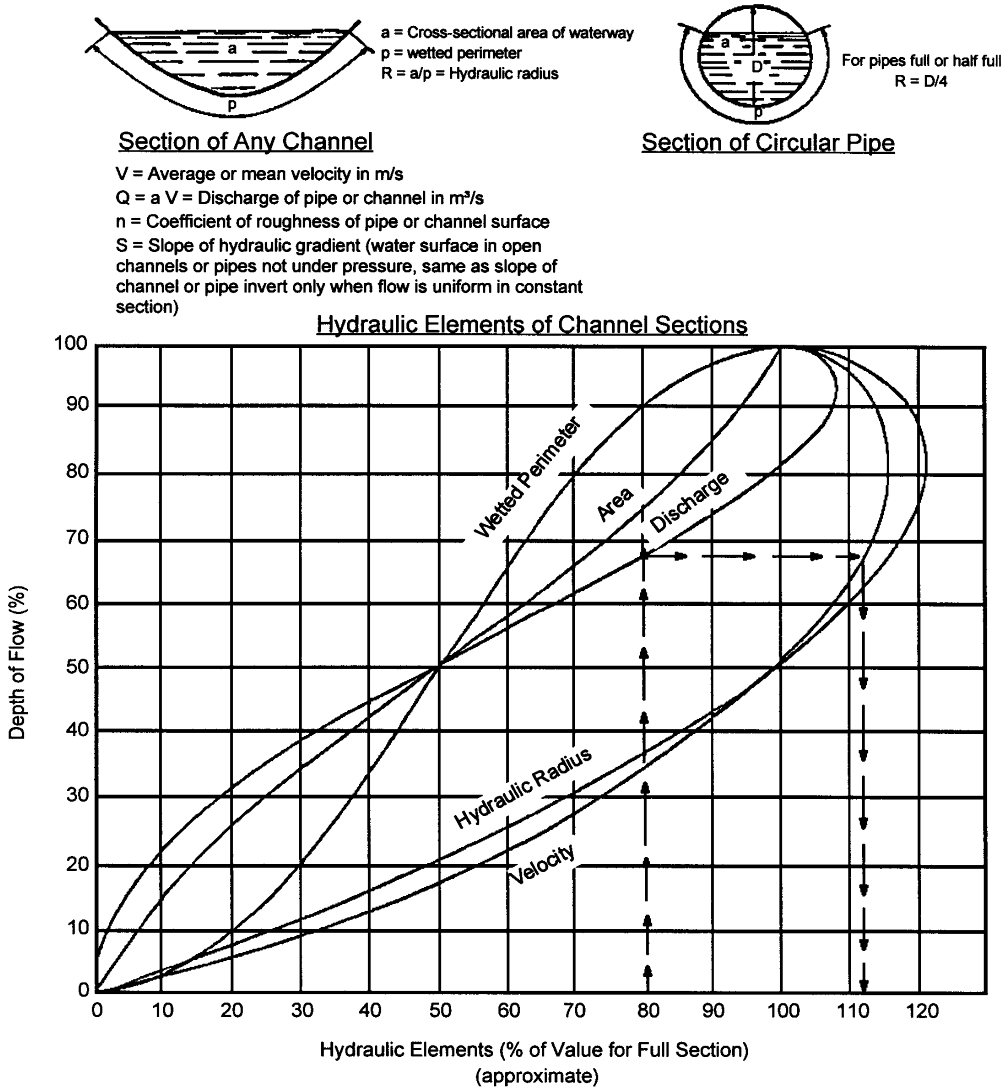


Figure 13-2 Values Of Hydraulic Elements Of Circular Section

For Various Depths Of Flow

13.3 Design Concepts (continued)

13.3.2 Minimum Grade

For very flat grades the general practice is to design components so that flow velocities will increase progressively throughout the length of the pipe system. The storm drainage system should be checked to be sure there is sufficient velocity in all of the drains to deter settling of particles. It is desirable that all storm drains be designed such that velocities of flow will not be less than 3 ft/sec at design flow. Minimum slopes required for a velocity of 3 ft/sec can be calculated by the Manning's formula or use values given in Table 13-1.

$$S = [Vn/0.59D^{0.67}]^2 \tag{13.6}$$

For concrete pipe, n=0.012 and a velocity of 3 ft/sec, this may be rewritten as

$$S = (0.003723)/D^{1.33} \tag{13.7}$$

Table 13-1

Minimum Slopes Necessary To Ensure v=3 ft/sec

In Storm Drains Flowing Full

Pipe Size, in.	Q, Full pipe ft^3/sec	Minimum Slopes ft/ft n = 0.012	n = 0.014	n = 0.024
18	5.30	0.0022	0.0030	0.0087
21	7.22	0.0018	0.0024	0.0071
24	9.43	0.0015	0.0020	0.0059
27	11.93	0.0013	0.0017	0.0051
30	14.73	0.0011	0.0015	0.0044
33	17.82	0.00097	0.0013	0.0039
36	21.21	0.00086	0.0012	0.0034
42	28.86	0.00070	0.00095	0.0028
48	37.70	0.00059	0.00080	0.0023
54	47.71	0.00050	0.00068	0.0020
60	58.90	0.00044	0.00059	0.0017
66	71.27	0.00038	0.00052	0.0015
72	84.82	0.00034	0.00046	0.0014

13.3 Design Concepts (continued)

13.3.3 Access Structures

13.3.3.1 Location

Access structures are utilized to provide entry to continuous underground storm drains for inspection and cleanout. In some locations grate inlets are used in lieu of access structures, when entry to the system can be provided at the inlet, so that the benefit of extra stormwater interception can be achieved with minimal additional cost. Typical locations where access structures should be specified are:

- where more than two storm drains converge,
- at intermediate points along tangent sections,
- where pipe size changes,
- where an abrupt change in horizontal or vertical alignment occurs
- where an abrupt change in elevation occurs.

Access structures should not be located in traffic lanes; however, when it is impossible to avoid locating an access structure in a traffic lane, care should be taken to insure it is not in the normal vehicle wheel path.

13.3.3.2 Spacing

The spacing of access structures should be in accordance with the following criteria:

Table 13-2

Pipe Size, inches	Maximum Spacing, ft.
Under 33”	330’
33 in. to 39 in.	440’
42 in. to 69 in.	660’
72 in. or greater	1200’

13.3.3.3 Sizing

ADOT C-Standards present details for normal manhole applications. When determining the minimum base required for various pipe sizes and locations, two general criteria must be met.

- The base structure must be large enough to accept the maximum pipe.
  A minimum width of D+3.5 feet is required.

- Access structure must be large enough to provide a minimum space between pipes. If pipes are located at substantially different elevations, pipes may not conflict and the above analysis is unnecessary. A minimum spacing of 12 inches shall be provided between the exterior walls of adjacent pipes.

13.3 Design Concepts (continued)

13.3.4 Curved Alignment

Curved storm drains are permitted where necessary. Long radius bend sections are available from many suppliers and are the preferable means of changing direction in pipes 48 inches and larger. Short radius bend sections are also available and can be utilized if there isn't room for the long radius bends. Deflecting the joints to obtain the necessary curvature is not desirable except in very minor curvatures. A deflection of 1.5 degrees is often possible at t joint, the designer should check with the pipe manufacturer for specifics regarding the proposed pipe sizes. Utilizing large access holes solely for changing direction may not be cost effective on large size storm drains.

13.3.5 Inverted Siphons

An inverted siphon carries the flow under an obstruction such as sanitary sewers, water mains, or any other structure or utility lane that is in the path of the storm drain line. The storm drain invert is lowered at the obstacle and is raised again after the crossing. The criteria for designing inverted siphons can be found in most of the hydraulics textbooks.

13.3.6 Time of Concentration

Pipe Sizing

The time of concentration for pipe sizing is defined as the time required for water to travel from the most hydraulically distant point of the watershed to the point of the storm drain system under consideration. It generally consists of two components: (1) the time to flow to the inlet which can consist of overland and channel or gutter flow and (2) the time to flow through the storm drain to the point under consideration.

Travel time within the storm drain pipes can be estimated by the relation:

$$t_t = L / 60V \tag{13.9}$$

Where: t_t = travel time, min
L = length of pipe in which runoff must travel, ft
V = estimated or calculated normal velocity, ft/sec

To summarize, the time of concentration for any point on a storm drain is the inlet time for the inlet at the upper end of the line plus the time of flow through the storm drain from the upper end of the storm drain to the point in question. In general, where there is more than one source of runoff to a given point in the storm drainage system, the longest t_c is used to estimate the intensity (I).

13.3 Design Concepts (continued)

Pipe Sizing (continued)

There could be exceptions to this generality, for example where there is a large inflow area at some point along the system, the t_c for that area may produce a larger discharge than the t_c for the summed area with the longer t_c. The designer should be cognizant of this possibility when joining drainage areas and determine which drainage area governs. To determine which drainage area controls, compute the peak discharge for each t_c. Note that when computing the peak discharge with the shorter t_c, not all the area from the basin with the longest t_c will contribute runoff. One way to compute the contributing area, A_c, is as follows:

$$\mathbf{A_c = A\,[t_{c1} / t_{c2}]} \qquad \textbf{(13.10)}$$

Where: $t_{c1} < t_{c2}$ and A is the area of the basin with the longest t_c.

13.4 Energy Losses

13.4.1 System Performance

The performance of a storm drain system is evaluated by determining the location of the hydraulic grade line during the design storm event. The hydraulic grade line is the location of the piezometric surface along the storm drain system. Usually it is helpful to **compute the EGL first, then subtract the velocity head (V^2/2g) to obtain the HGL.** Water flow is driven by the difference in energy head from one location to another. As water flows it encounters many obstructions that cause a loss of energy. The computation of the EGL requires the evaluation of the energy losses in the system.

13.4.2 Tailwater

For most design applications, **the tailwater will either be above the crown of the outlet** or can be considered to be between the crown and critical depth. To determine the EGL, begin with the tailwater elevation or $(d_c + D)/2$, whichever is higher, **add the velocity head for full flow** and proceed upstream to compute all losses such as exit losses, friction losses, junction losses, bend losses and entrance losses as appropriate.

An exception to the above might be a very large outfall with low tailwater when a water surface profile calculation would be appropriate to determine the location where the water surface will intersect the top of the barrel and full flow calculations can begin. In this case, the downstream water surface elevation would be based on critical depth or the tailwater, whichever is higher.

When estimating tailwater depth on the receiving stream, the prudent designer will consider the joint or coincidental probability of two events occurring at the same time. For the case of a tributary stream or a storm drain, its relative independence may be qualitatively evaluated by a comparison of its drainage area with that of the receiving stream. A short duration storm that causes peak discharges on a small basin may not be critical for a larger basin. Also, it may safely be assumed that if the same storm causes peak discharges on both basins, the peaks will be out of phase. To aid in the evaluation of joint probabilities, refer to the Table below.

Table 13-3

AREA RATIO	FREQUENCIES FOR COINCIDENTAL OCCURRENCE			
	10-Year Design		100- Year Design	
	Mainstream	Tributary	Mainstream	Tributary
10,000 to 1	1 10	10 1	2 100	100 2
1,000 to 1	2 10	10 2	10 100	100 10
100 to 1	5 10	10 5	25 100	100 25
10 to 1	10 10	10 10	50 100	100 50
1 to 1	10 10	10 10	100 100	100 100

Reference: USCE, Norfolk District, 1974

13.4 Energy Losses (continued)

13.4.3 Exit Losses

The exit loss is a function of the change in velocity at the outlet of the pipe. For a sudden expansion such as an endwall, the exit loss is:

$$H_o=1.0[V^2/2g - V_d^2/2g] \qquad \textbf{(13.11)}$$

Where: V = average outlet velocity, ft/sec
V_d = channel velocity downstream of outlet, ft/sec

Note that when V_d = 0 as in a reservoir, the exit loss is one velocity head. For part full flow where the pipe outlets in a channel with moving water, the exit loss may be reduced to virtually zero.

13.4.4 Bend Losses

For bends outside of structures, the bend loss coefficient for storm drain or large radii is minor but can be evaluated using the formula:

$$h_b = 0.0033\ (\Delta)\ (V_o^2 / 2g) \qquad \textbf{(13.12)}$$

Where: Δ = angle of curvature in degrees, less than or equal to 90 degrees.

13.4.5 Pipe Friction Losses

The friction slope is the energy gradient in ft/ft for that run. The friction loss is simply the energy gradient multiplied by the length of the run in feet. Energy losses from pipe friction may be determined by rewriting the Manning's equation with terms as previously defined:

$$S_f = [Qn/1.486\ AR^{2/3}]^2 = [Qn/0.46\ D^{2/3}]^2 \qquad \textbf{(13.13)}$$

$$\text{or}\ \ S_f = [Vn/0.59D^{0.67}]^2 \qquad \textbf{(13.14)}$$

The head losses due to friction may be determined by the formula:

$$H_f=L*S_f \qquad \textbf{(13.15)}$$

The Manning's equation can also be written to determine friction losses for storm drains as follows:

$$H_f= \frac{V^2\ n^2\ L}{2.21gR^{4/3}} \qquad \textbf{(13.16)}$$

$$H_f= \frac{V^2\ n^2\ L}{0.35\ D^{4/3}} \qquad \textbf{(13.17)}$$

13.4 Energy Losses (continued)

13.4.5 Pipe Friction Losses (continued)

Where: H_F = total head loss due to friction, ft
n = Manning's roughness coefficient
D = diameter of pipe, ft
L = length of pipe, ft
V = mean velocity, ft/sec
R = hydraulic radius, ft
g = 32.2 ft/sec^2
S_f = slope of hydraulic grade line, ft/ft

13.4.6 Expansion and Contraction losses

Pipe size transitions are sometimes made without the use of a junction structure. The energy loss is taken as $h = k*\Delta V^2/2g$

Values of K are in the tables in Appendix E.

Open Channel Flow

Expansion

$$h = \frac{k_e(V_1^2 - V_2^2)}{2g}$$

Contraction

$$h = \frac{k_c*(V_2^2 - V_1^2)}{2g}$$

Pressure Flow

$$h = \frac{k*(V_2^2)}{2g}$$

13.4.7 Structure Losses

There are two types of structures to consider for structure losses. Pipes may be connected using manufactured wyes or tees or the may be connected at access structures such as manholes. These two types of structures have different losses and different methods for estimating those losses.

Manufactured Wye or Tee Losses.
The loss at a manufactured wye or tee is

$$Hj = \frac{(Q_oV_o) - (Q_iV_i) - Q_lV_l\cos(\square)}{(0.5\,(A_o + A_i))\quad g} + \frac{V_i^2 - V_o^2}{2g} \tag{13.18}$$

Where:
Q_o = outlet discharge, cfs
V_o= outlet velocity, ft/sec
A_o= outlet cross-sectional area, sq. ft.

13.4 Energy Losses (continued)

13.4.7 Structure Losses (continued)

Q_i= inlet discharge, cfs
V_i= inlet velocity, ft/sec
A_i= inlet cross-sectional area, sq.ft.

Q_l= connector/lateral pipe discharge, cfs
V_l= connector/lateral pipe velocity, ft/sec.
□= angle between the inflow and outflow pipes.

Access Structure Losses
The head loss encountered in going from one pipe to another through an access hole is commonly represented as being proportional to the velocity head at the outlet pipe. Using K to signify this constant of proportionality, the energy loss is approximated as $H_j = K \times (V_o^2/2g)$. Experimental studies have determined that the K value can be approximated as follows:

$$K = K_o C_D C_d C_Q C_p C_B \qquad \textbf{(13.19)}$$

Where: K = adjusted loss coefficient
K_o = initial head loss coefficient based on relative access hole size.
C_D = correction factor for pipe diameter (pressure flow only)
C_d = correction factor for flow depth (non-pressure flow only)
C_Q = correction factor for relative flow
C_B = correction factor for benching
C_p = correction factor for plunging flow

Relative Structure Hole Size and Angle of Deflection

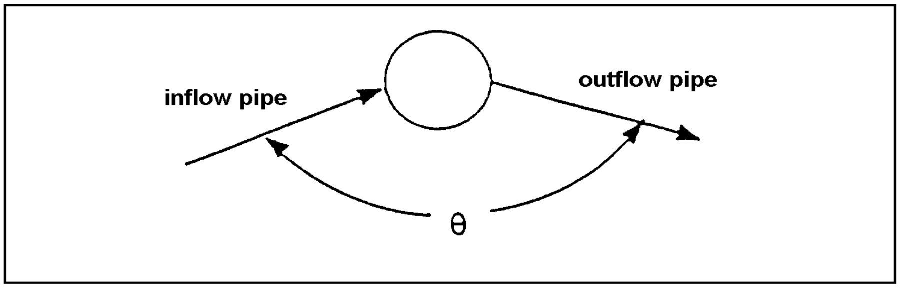
K_o is estimated as a function of the relative access hole size and the angle of deflection between the inflow and outflow pipes.

$$K_o = 0.1(b/D_o)(1-\sin(□)) + 1.4(b/D_o)^{0.15} \sin(□) \qquad \textbf{(13.20)}$$

Where: □ = the angle between the inflow pipe under consideration and the outflow pipe.
b = access hole diameter, ft
D_o = outlet pipe diameter, ft

13.4 Energy Losses (continued)

13.4.7 Structure Losses (continued)



Deflection Angle

If the structure is a rectangular box, it should be treated as an equivalent circular structure.

Pipe Diameter

A change in head loss due to differences in pipe diameter is **only significant in pressure flow situations when the depth in the access hole to outlet pipe diameter ratio, d/D_o, is greater than 3.2**. Therefore, it is only applied in such cases.

$$C_D = (D_o / D_i)^3 \tag{13.21}$$

Where: D_i = incoming pipe diameter, ft
D_o = outgoing pipe diameter, ft

Flow Depth

The correction factor for flow depth is significant only in cases of **free surface flow or low pressures, when d/D_o ratio is less than 3.2 and is only applied in such cases.** Water depth in the access hole is approximated as the level of the hydraulic gradeline at the upstream end of the outlet pipe.

The correction factor for flow depth, C_d, is calculated by the following:

$$C_d = 0.5(d/D_o)^{0.6} \tag{13.22}$$

Where: d = water depth in access hole above invert of outlet pipe, ft
D_o= outlet pipe diameter, ft

13.4 Energy Losses (continued)

13.4.7 Structure Losses (continued)

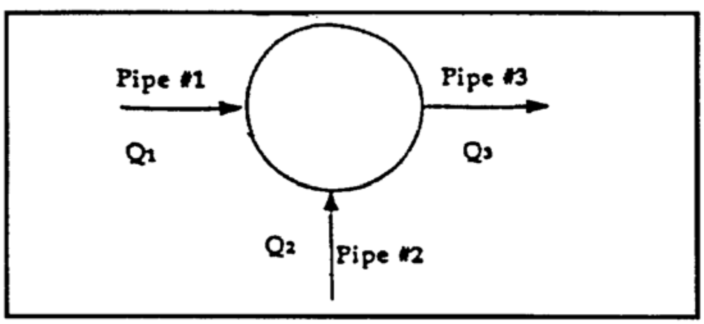
Relative Flow

The correction factor for relative flow, C_Q, is a function of the angle of the incoming flow as well as the percentage of flow coming in through the pipe of interest versus other incoming pipes. It is computed as follows:

$$C_Q = (1 - 2\sin(\square))(1 - Q_i/Q_o)^{0.75} + 1 \qquad \textbf{(13.23)}$$

where: C_Q = correction factor for relative flow
$\square$ = the angle between the inflow and outflow pipes
Q_i = flow in the inflow pipe under consideration, ft^3/sec
Q_o = flow in the outlet pipe, ft^3/sec

As can be seen from the equation, C_Q is a function of the angle of the incoming flow as well as the percentage of flow coming in through the pipe of interest versus other incoming pipes. To illustrate this effect, consider the access structure shown in the Figure and assume the following two cases to determine the impact flows of pipe 2 entering the access hole.



Relative Flow Effect

Case 1:
Q_1= 3.3 ft^3/sec, Q_2= 1.1 ft^3/sec, Q_3= 4.4 ft^3/sec

$C_{3\text{-}1} = (1\text{-}2^*\sin(180°))(1 - 3.3/4.4)^{0.75} + 1$
$C_{3\text{-}1} = (1\text{-}0)(1\text{-}0.75)^{0.75} + 1 = 1.35$

$C_{2\text{-}1} = (1\text{-}2^*\sin(90°))(1 - 1.1/4.4)^{0.75} + 1$
$C_{2\text{-}1} = (1\text{-}2)(1\text{-}0.25)^{0.75} + 1 = 0.19$

Case 2:
Q_1= 1.1 ft^3/sec, Q_2=3.3 ft^3/sec, Q_3= 4.4 ft^3/s

$C_{3\text{-}1} = (1\text{-}2^*\sin(180°))(1 - 1.1/4.4)^{0.75} + 1$
$C_{3\text{-}1} = (1\text{-}0)(1\text{-}0.25)^{0.75} + 1 = 1.81$

$C_{2\text{-}1} = (1\text{-}2^*\sin(90°))(1 - 3.3/4.4)^{0.75} + 1$
$C_{2\text{-}1} = (1\text{-}2)(1\text{-}0.75)^{0.75} + 1 = 0.65$

13.4 Energy Losses (continued)

13.4.7 Structure Losses (continued)

Plunging Flow

This correction factor corresponds to the effect of another inflow pipe or surface flow from an inlet, plunging into the access hole, on the inflow pipe for which the head loss is being calculated. Using the notations in the figure below for the example, C_p is calculated for pipe # 1 when pipe # 2 discharges plunging flow. **The correction factor is only applied when h > d.**

The correction factor for plunging flow, C_p, is calculated by the following:

$$C_p = 1+0.2[h/ D_o][(h- d)/ D_o] \qquad \textbf{(13.24)}$$

Where: C_p = correction for plunging flow
h = vertical distance of plunging flow from flow line of incoming pipe to the center of outlet pipe, ft
D_o = outlet pipe diameter, ft
d = water depth in access hole, ft

Benching

Normally, the bottom of an access structure constructed in accordance with C-18.10 can be considered to be full-round. The correction for benching in the access hole, C_B, is obtained from Table 13-8. Benching tends to direct flows through the access hole, resulting in reductions in head loss. For flow depths between the submerged and unsubmerged conditions, a linear interpolation is performed.

Losses at inlets

At open inlets to a storm drain system, an inlet will function the same as a culvert inlet. Under inlet control, the hydraulic grade line at the entrance is estimated by using the inlet control coefficients presented in the Culvert Chapter. For inlets operating under outlet control, entrance losses can be calculated using

$$h_i = k_e (V^2/2g) \qquad \textbf{(13.25)}$$

Where: h_i = headloss at inlet, ft.
k_e = entrance loss coefficient. Values are listed in the Culvert Chapter.

Summary

In summary, to estimate the head loss through an access structure from the outflow pipe to a particular inflow pipe, multiply the above correction factors together to get the head loss coefficient, K. This coefficient is then multiplied by the velocity head in the **outflow pipe** to estimate the minor loss for the connection.

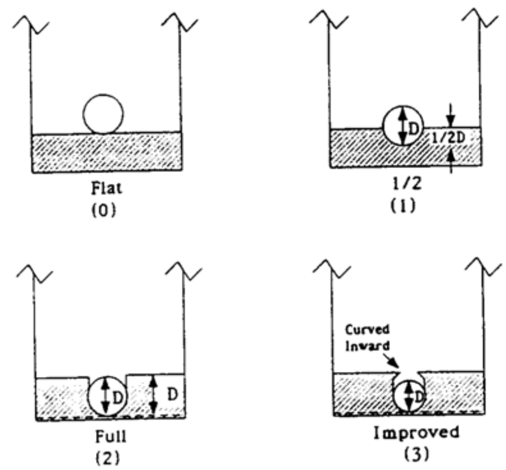
13.4 Energy Losses (continued)

13.4.7 Structure Losses (continued)

Schematic Representation Of Benching Types

Table 13-4

Bench Type	Correction Factors, C_B	
	Submerged*	Unsubmerged**
Flat floor	1.00	1.00
Half Bench	0.95	0.15
Full Bench	0.75	0.07
Improved	0.40	0.02
*pressure flow, $d/D_o > 3.2$ **free surface flow, $d/D_o < 1.0$		



13.5 Design Procedure

The design of storm drainage systems is generally divided into the following operations:

- Storm drain design computation should be documented on forms. A sample is as illustrated in Figure 13-4.

Step 1 Determine inlet location and spacing as outlined in Chapter 12.

Step 2 Prepare plan layout of the storm drainage system establishing the following design data:
a. Location of storm drains.
b. Direction of flow.
c. Location of access holes.
d. Location of existing utilities such as water, gas, or underground cables.

Step 3 Set a preliminary slope between the outfall and the upstream end of the system. On a profile, plot the outfall elevations, and the invert elevation of any inlet that would control the invert elevations of the storm drain.

Step 4 From the inlet design get the drainage areas and runoff coefficient. For the first inlet in the system get the time of concentration to the first inlet. Using an Intensity-Duration-Frequency (IDF) curve, determine the rainfall intensity. Calculate the discharge by multiplying $C \times I \times A$.

Step 5 Size the pipe to convey the discharge assuming full flow by varying the slope and pipe size as necessary.

Step 6 Calculate travel time in the pipe to the next inlet or access hole by dividing pipe length by the velocity. This travel time is added to the time of concentration for a new time of concentration and a new rainfall intensity at the next entry point.

Step 7 At the next inflow point, calculate the additional CA, the sub area (A) multiplied by the subarea runoff coefficient (C), then add to the previous CA. Multiply the total CA by the rainfall intensity at the computed time of concentration to determine the new discharge. If the local CA is large, compare this discharge with the CA for the subbasin and the rainfall intensity using the inlet time of concentration. Use the larger of the two discharges. Determine the size of pipe and slope necessary to convey the discharge.

Step 8 Continue this process to the storm drain outlet. Utilize the equations and/or nomographs to accomplish the design effort.

Step 9 Complete the design by calculating the hydraulic grade line as described in Section 13.6.

13.5 Design Procedure (continued)

Storm Drain Computation Sheet

Computed ______ Date ______

Checked ______ Date ______

Route ______

Section ______

County ______

Station		Length (m)	Drainage Area A (ha)		Runoff Coefficient C	A x C		Flow Time (min)		Rainfall Intensity I (mm/h)	Total Runoff 0.0278 CIA = Q (m^3/s)	Diameter Pipe (mm)	Capacity Full (m^3/s)	Velocity (m/s)		Inert Elev.		Manhole Invert Drop	Slope of Drain (m/m)
From	To		Increment	Total		Increment	Total	To Upper End	In Section					Flowing Full	Design Flow	Upper End	Lower End		

Figure 13-4 Storm Drain Computation Sheet

13.6 Hydraulic Grade Line

13.6.1 Introduction

The hydraulic grade line (HGL) is the last feature to be established relating to the hydraulic design of storm drains. The hydraulic grade line guides the designer in determining the acceptability of the proposed system by establishing the elevations along the system to which the water will rise when the system is operating at a design frequency flow discharge. A special concern with storm drains designed to operate under pressure flow conditions is that inlet surcharging and possible access structure lid displacement can occur if the hydraulic grade line rises above the ground surface. A design based on open channel conditions must be carefully planned as well, including evaluation of the potential for excessive and inadvertent flooding created when a storm event larger than the design storm pressurizes the system. As hydraulic calculations are performed, frequent verification of the existence of the desired flow condition should be made. Storm drain systems can often alternate between pressure and open channel flow conditions from one section to another.

The calculation of the hydraulic grade line is usually for subcritical flow. Therefore, the computation begins at the system outfall with the tailwater elevation and a known HGL. If the outfall is an existing storm drain system, the EGL calculation must begin at the outlet end of the existing system and proceed upstream through this in-place system, then upstream through the proposed system. To the known energy grade is added the pipe friction losses, resulting in the energy grade line at the upstream end of the pipe. Adding the energy losses in the junction gives the energy grade line for the downstream end of the upstream pipes.

C**ompute the EGL first, then subtract the velocity head ($V^2/2g$) to obtain the HGL.** See Figure 9-4 for a sketch of a culvert outlet that depicts the difference between the HGL and the energy grade line (EGL). In general, if the HGL is above the crown of the pipe, pressure flow hydraulic calculations are appropriate. Conversely, if the HGL is below the crown of the pipe, open channel flow calculations are appropriate. Open channel flow should be assumed only when the flow depth is less than 80% of the pipe diameter. The design should normally be sized assuming full flow and normally analyzed for pressure flow. Flow events greater than the design event will result in pressure flow. The process is repeated throughout the storm drain system. At each junction the HGL should be checked versus the inlet or ground elevation. If all HGL elevations are acceptable then the hydraulic design is adequate. If the HGL exceeds the desirable elevation, then adjustments to the design must be made to lower the water surface elevation. See Figure 13-18 for a sketch depicting the results of and inadequate designs of a storm drain system.

13.6.2 HGL Calculation Procedure

A step-by-step procedure necessary to manually calculate the location of the hydraulic gradeline, HGL, is presented below. The procedure can be documented by the use of Table 13-5.

Step 1 Enter in Col. 1 the station for the junction immediately upstream of the outflow pipe. EGL computations begin at the outfall and are worked upstream taking each junction into consideration.

Step 2 Enter in Col. 2 the tailwater elevation, refer to Section 13.4.2 for procedure.

Step 3 Enter in Col. 3 the diameter (D_o) of the outflow pipe.

13.6 Hydraulic Grade Line

13.6.2 HGL Calculation Procedure (continued)

Step 4 Enter in Col. 4 the design discharge (Q_o) for the outflow pipe.

Step 5 Enter in Col. 5 the length, L_o, of the outflow pipe.

Step 6 Enter in Col. 6 the outlet velocity of flow, V_o.

Step 7 Enter in Col. 7 the velocity head, $V_{o2}/2g$.

Step 8 Enter in Col. 8 the exit loss, H_o.

Step 9 Enter in Col. 9 the friction slope (S_{fo}) in ft/ft of the outflow pipe. This can be determined by using the equation 13.13. **Note: Assumes full flow conditions.**

Step 10 Enter in Col. 10 the friction loss (H_f) that is computed by multiplying the length (L_o) in Col. 5 by the friction slope (S_{fo}) in Col 9. On curved alignments, calculate curve losses by using the formula H_c = 0.0033 (Δ)($V_o^2/2g$), where Δ = angle of curvature in degrees, and add to the friction loss.

Step 11 If the connection of pipes is made with a wye or tee, enter n/s in columns 11 through 17 and enter H_j in column 18 as computed by equation 13.18, then go to step 19 or if the connection is made with an access structure, enter in Col. 11 the initial head loss coefficient, K_o, based on relative access hole size as computed by equation 13.20.

Step 12 Enter in Col. 12 the correction factor for pipe diameter, C_D, as computed by equation 13.21.

Step 13 Enter in Col. 13 the correction factor for flow depth, C_d, as computed by equation 13.22. Note this factor is only significant in cases where the d/D_o ratio is less than 3.2.

Step 14 Enter in Col. 14 the correction factor for relative flow, C_Q, as computed by equation 13.23.

Step 15 Enter in Col. 15 the correction factor for plunging flow, C_p, as computed by equation 13.24. The correction factor is only applied when h>d.

Step 16 Enter in Col. 16 the correction factor for benching, C_B, as determined in Table 13-4.

Step 17 Enter in Col. 17 the value of K as computed by equation 13.19.

Step 18 Enter in Col. 18 the value of the total access hole loss, $K\,V_o^2/2g$.

Step 19 If the tailwater submerges the outlet end of the pipe, enter in Col. 19 the sum of Col. 2 (TW elevation) and Col. 7 (exit loss) to get the EGL at the outlet end of the pipe. If the pipe is flowing full, but the tailwater is low, the EGL will be determined by adding the velocity head to (d_c + D)/2.

13.6 Hydraulic Grade Line

13.6.2 HGL Calculation Procedure (continued)

Step 20 Enter in Col. 20 the sum of the friction head (Col 10), the access hole losses (Col 18), and the energy grade line (Col 19) at the outlet to obtain the EGL at the inlet end. This value becomes the EGL for the downstream end of the upstream pipe.

Step 21 Determine the HGL (Col 21) throughout the system by subtracting the velocity head (Col 7) from the EGL (Col 20).

Step 22 Check to make certain that the HGL is below the level of allowable high water at that point. If the HGL is above the finished grade elevation, water will exit the system at this point for the design flow.

Connector pipe profiles will usually have an entrance loss equal to a pipe culvert, $(1+k_e)*(V_1^2/2g.)$

The above procedure applies to pipes that are flowing full, as should be the condition for design of new systems. If a part full flow condition exists, the EGL is located one velocity head above the water surface.

13.6 Hydraulic Grade Line

13.6.2 HGL Calculation Procedure (continued)

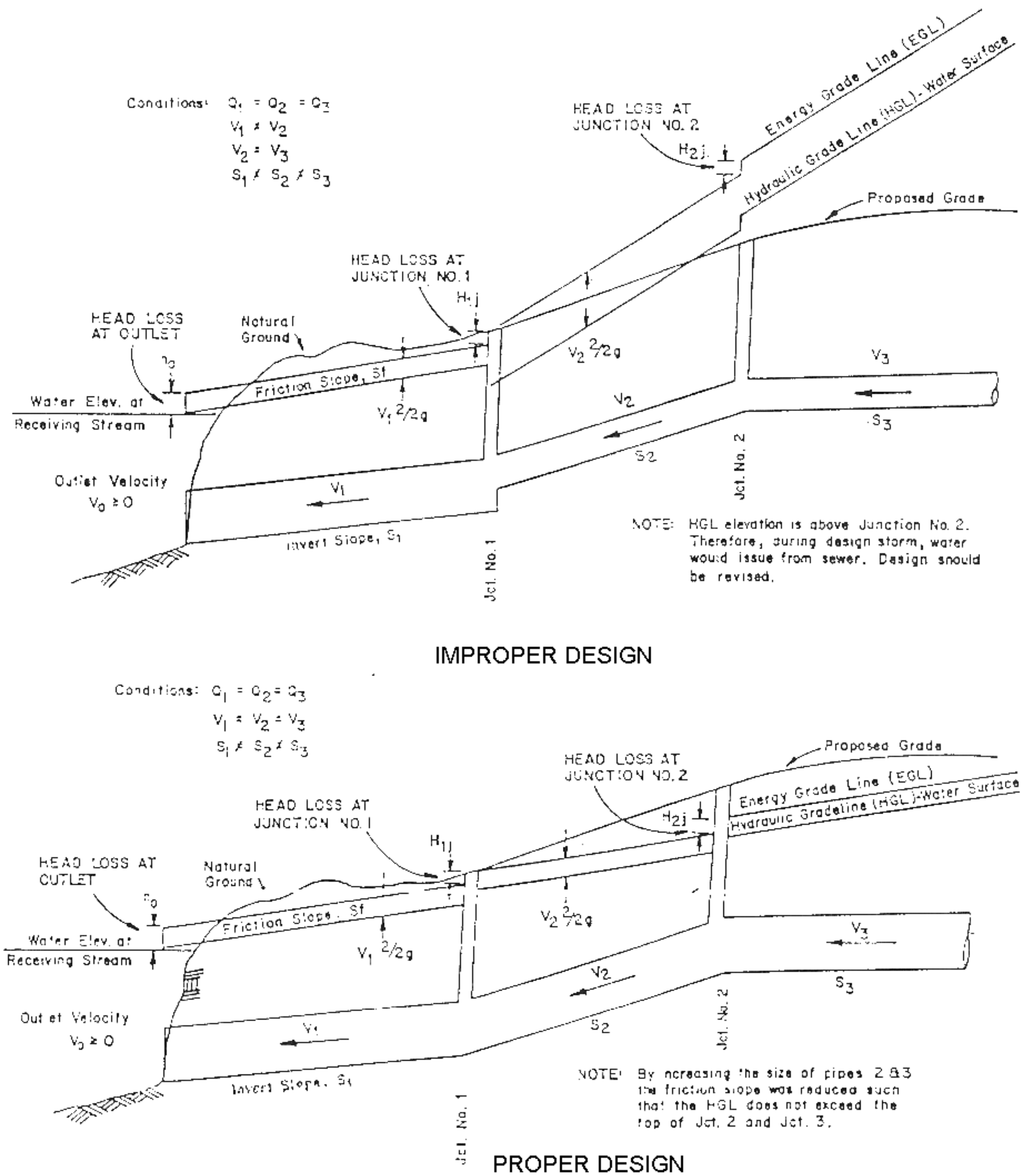


Figure 13-5 Use Of Energy Losses In Developing a Storm Drain System

13.6 Hydraulic Grade Line

13.6.2 HGL Calculation Procedure (continued)

Table 13-5 Hydraulic Grade Line Computation Form

Station	TW	D_0	Q_o	L_o	V_o	$V_o^2/2g$	H_o	S_{fo}	H_f	K_o	C_D	C_d	C_Q	C_p	C_B	K	$K(V_o^2/2g)$	EGL_0 2+7	EGL_i 10+18-+19	HGL EGL-7	TOC Elev
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)

13.7 Computer Programs

To assist with storm drain system design, many computer programs have been developed for the computation of hydraulic grade line. These computer programs can be used to check design adequacy and to analyze the performance of a storm drain system under assumed inflow conditions. The user must understand how the computer program calculates

- the time of concentration,
- how it interpolates the rainfall intensity,
- whether it uses the full flow or design flow velocity for travel time computations
- how it calculates the junction losses

Appendix 13.A presents an example problem using manual methods. It may be used to evaluate computer programs

13.8 References

American Association of State Highway and Transportation Officials. Volume 9, Highway Drainage Guidelines, Storm Drainage Systems. 1992.

Bridge Deck Drainage Guidelines. FHWA Report No. RD-014. December 1986.

Federal Highway Administration. Drainage of Highway Pavements, Hydraulic Engineering Circular No. 22.

Federal Highway Administration. Design of Bridge Deck Drainage, Hydraulic Engineering Circular No. 21. 1993.

Federal Highway Administration. Design of Urban Highway Drainage — The State of The Art, FHWA-TS-79-225. 1979.

Federal Highway Administration. Pavement and Geometric Design Criteria For Minimizing Hydroplaning. FHWA Report No. RD-79-31. December 1979.

Dah-Chen Woo. Public Roads, Vol.52, No. 2 Bridge Drainage System Needs Criteria. U. S. Department of Transportation. September 1988.

Appendix 13.A Example Problem

Storm Drain Example: Crandall Boulevard

Determine the appropriate pipe sizes and profile for the system. Evaluate the HGL.
Use concrete pipe with a Manning's n of 0.013, a minimum pipe diameter of 18 inches.
Tabled below is the data for the system.
Drainage area information is in Table 13-8.1
Rainfall information is in Table 13-8.2
Pipe geometric data is in Table 13-8.3

Step 1. Prepare preliminary layout and number the pipes. Use numbering system from outfall to upstream. Determine the Design Discharge and initial pipe sizes.

TABLE 13-A.1 Drainage basin data.

Inlet No.	Inlet Station	Drainage Area, Acre	"C"	Effective Area "CA"	Time of Concentration, min.
A (Pipe 8)	101+00	6.48	0.51	3.30	10
B (Pipe 9)	102+40	4.95	0.53	2.62	12
C (Pipe 10)	102+40	4.39	0.49	2.15	12
D (Pipe 11)	103+70	3.76	0.53	1.99	10
E (Pipe 12)	105+00	11.03	0.62	6.84	15
F (Pipe 13)	107+80	13.54	0.59	7.99	18
G (Pipe 14)	B106+25	7.42	0.58	4.30	16
H (Pipe 15)	B108+75	6.2	0.8	4.96	12

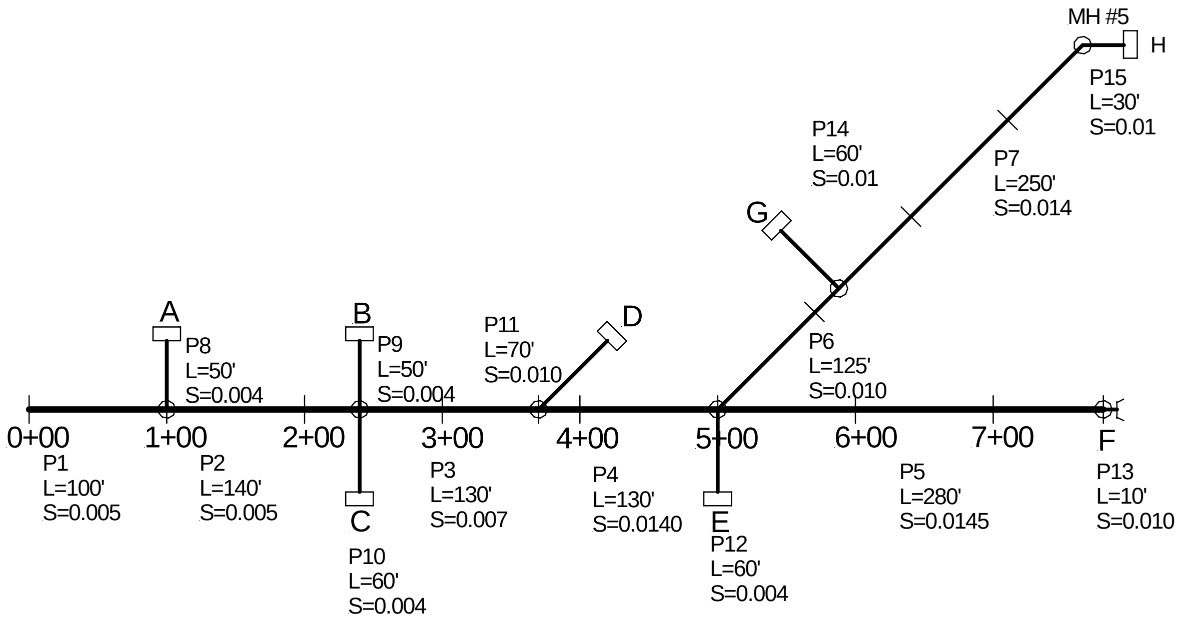
TABLE 13-A.2 Rainfall Data

Time, Min	10	15	20	30	40	50	60
Intensity (in/hr)	4.80	3.96	3.33	2.76	2.25	1.85	1.55

Appendix 13.A Example Problem (continued)

TABLE 13-A.3 Pipe Data

Pipe ID	Diameter, Ft.	Slope, ft/ft	Length, Ft	D/S Invert	U/S Invert
1	48	0.005	100	101.00	101.50
2	48	0.005	140	101.50	102.20
3	48	0.007	130	102.20	103.11
4	42	0.0140	130	103.61	105.43
5	36	0.0145	280	105.93	109.99
6	30	0.010	125	106.43	107.68
7	24	0.014	250	108.18	111.68
8	24	0.004	50		
9	24	0.004	50		
10	24	0.004	60		
11	24	0.01	70		
12	24	0.004	60		
13	36	0.01	10	109.99	110.09
14	24	0.01	60		
15	24	0.01	30	111.68	111.98



STORM DRAIN LAYOUT

Appendix 13.A Example Problem (continued)

Step 2. Determine the Design Discharge and initial pipe sizes.

Begin at upstream of line "B"

Pipe 7. L=250 ft., slope = 0.014
Area "H". CA=4.96.
Inlet Tc=12.0 min. Travel time in connector pipe = 0.06 min.
I=4.4 in/Hr. Q= 4.96*4.4 = 21.82 cfs.

Minimum pipe diameter for full flow:
$D=1.33[Qn/S^{0.5}]^{0.375}$

$D=1.33[21.82*0.013/(0.014)^{0.5}]^{0.375}$

D= 1.85 ft. use 24" pipe.

For 24" pipe, full flow capacity $Q=\frac{0.46\ D^{2.67}S^{0.5}}{n} = \frac{0.46\ (2.0)^{2.67}*(0.014)^{0.5}}{0.013} = 26.65$ cfs

Full flow velocity $V=\frac{0.59\ D^{0.67}S^{0.5}}{n} = \frac{0.59\ (2.0)^{0.67}*(0.014)^{0.5}}{0.013} = 8.48$ ft/sec

From chart 13-18, determine design velocity: Q/Qf=21.82/26.65=0.82, therefore V/Vf=1.13

Design velocity = 1.13*8.48=9.58 ft/sec = 575 ft/min.

Travel time = L/V = (250/575) =0.43 min.

Pipe 6. L=125 ft., slope = 0.01
Area "G"
Additional Ca= 4.30. Total CA=9.26
Inlet Tc=16 min. Travel time in connector pipe = 0.06 min.
System Tc=12.06 + 0.43 = 12.49 minutes, use Tc of 16.06 minutes.
I= 3.82 in/hr. Q=9.26* 3.82 = 35.39 cfs.

Minimum pipe diameter for full flow:
$D=1.33[Qn/S^{0.5}]^{0.375}$

$D=1.33[35.39*0.013/(0.01)^{0.5}]^{0.375}$

D= 2.36 ft., use 2.5' pipe.

For 30" pipe, full flow capacity $Q=\frac{0.46\ D^{2.67}S^{0.5}}{n} = \frac{0.46\ (2.5.5)^{2.67}*(0.010)^{0.5}}{0.013} = 40.86$ cfs

Appendix 13.A Example Problem (continued)

Step 2. Determine the Design Discharge and initial pipe sizes. (continued)

Full flow velocity $V = \frac{0.59\, D^{0.67} S^{0.5}}{n} = \frac{0.59\,(2.5)^{0.67} * (0.010)^{0.5}}{0.013} = 8.32$ ft/sec

From chart 13-18, determine design velocity: Q/Qf=35.39/40.86 = 0.87, therefore V/Vf=1.15

Design velocity = 1.15*8.32=9.57 ft/sec = 574 ft/min.

Travel time = L/V = (125/574) =0.22 min.

Pipe 5 Length =280, slope =0.0145
Area "F"
CA=7.99. Inlet Tc=18 min. Travel time in connector pipe = 0.06 min. Tc=18.06 min.
I=3.60 in/Hr. Q= 7.99*3.6=28.76 cfs.

Minimum pipe diameter for full flow:
$D=1.33[Qn/S^{\mathbf{0.5}}]^{\mathbf{0.375}}$

$D=1.33[28.76*0.013/(0.0145)^{\mathbf{0.5}}]^{\mathbf{0.375}}$

D= 2.03 ft., use 36" pipe. Based on allowable headwater for pipe culvert in a headwall.

For 36" pipe, full flow capacity $Q = \frac{0.46\, D^{2.67} S^{0.5}}{n} = \frac{0.46\,(3.0)^{2.67} * (0.0145)^{0.5}}{0.013} = 79.76$ cfs

Full flow velocity $V = \frac{0.59\, D^{0.67} S^{0.5}}{n} = \frac{0.59\,(3.0)^{0.67} * (0.0145)^{0.5}}{0.013} = 11.37$ ft/sec

From chart 13-18, determine design velocity: Q/Qf=28.76/79.76=0.36, therefore V/Vf=0.92

Design velocity = 0.92*11.37=10.46 ft/sec = 628 ft/min.

Travel time = L/V = (280/628) =0.45 min.

Pipe 4. Length= 130, slope = 0.0140
Area "E" Additional CA=6.84
Upstream end, CA= 9.26+7.99=17.25. Total CA=24.09
Inlet Tc= 15 min., Travel time in connector pipe = 0.06 min.
System Tc=18.51 use system Tc of 18.51 minutes. I=3.5 in/hr.
Q=24.09*3.50 = 84.32 cfs.

Minimum pipe diameter for full flow:
$D=1.33[Qn/S^{\mathbf{0.5}}]^{\mathbf{0.375}}$

$D=1.33[84.32*0.013/(0.0140)^{\mathbf{0.5}}]^{\mathbf{0.375}}$

Appendix 13.A Example Problem (continued)

Step 2. Determine the Design Discharge and initial pipe sizes. (continued)

Pipe 4. (continued)

D= 3.06 ft., use 3.5’ pipe.

For 42” pipe, full flow capacity $Q=\dfrac{0.46\ D^{2.67}S^{0.5}}{n} = \dfrac{0.46\ (3.5)^{2.67}*(0.0140)^{0.5}}{0.013} = 118.7$ cfs

Full flow velocity $V=\dfrac{0.59\ D^{0.67}S^{0.5}}{n} = \dfrac{0.59\ (3.5)^{0.67}*(0.0140)^{0.5}}{0.013} = 12.43$ ft/sec

From chart 13-18, determine design velocity: Q/Qf=84.32/118.7=0.71, therefore V/Vf=1.10

Design velocity = 1.10*12.43=13.67 ft/sec = 820 ft/min.

Travel time = L/V = (130/820) =0.16 min.

Pipe 3. Length = 130, slope = 0.007
Area “D” Additional CA=1.99

Total CA=24.09+1.99= 26.08 Ac.
Inlet Tc= 10.0 min., system Tc=18.51+0.16 = 18.67, use system Tc of 18.67 minutes.
I= 3.5 in/hr.
Q=26.08*3.5 = 91.29 cfs.

Minimum pipe diameter for full flow:
$D=1.33[Qn/S^{\mathbf{0.5}}]^{\mathbf{0.375}}$

$D=1.33[91.29*0.013/(0.007)^{\mathbf{0.5}}]^{\mathbf{0.375}}$

D= 3.59 ft, use 4.0’ pipe.

For 48” pipe, full flow capacity $Q=\dfrac{0.46\ D^{2.67}S^{0.5}}{n} = \dfrac{0.46\ (4.0)^{2.67}*(0.007)^{0.5}}{0.013} = 119.9$ cfs

Full flow velocity $V=\dfrac{0.59\ D^{0.67}S^{0.5}}{n} = \dfrac{0.59\ (4.0)^{0.67}*(0.007)^{0.5}}{0.013} = 9.54$ ft/sec

From chart 13-18, determine design velocity: Q/Qf=91.29/119.9=0.76, therefore V/Vf=1.13

Design velocity = 1.13*9.54=10.78 ft/sec = 647 ft/min.

Travel time = L/V = (130/647) =0.20 min.

Appendix 13.A Example Problem (continued)

Step 2. Determine the Design Discharge and initial pipe sizes. (continued)

Pipe 2: Length = 140, slope = 0.005
Area "B" & "C" Additional CA= 2.62+2.15 = 4.77 Ac.
Total CA= 26.08+4.77=30.85

Inlet Tc= 12 min. system time = 18.67 + 0.20= 18.87. use system time
I= 3.49 in/hr.
Q=30.85*3.49=107.7 cfs.

Minimum pipe diameter for full flow:
$D=1.33[Qn/S^{\mathbf{0.5}}]^{\mathbf{0.375}}$

$D=1.33[107.7*0.013/(0.005)^{\mathbf{0.5}}]^{\mathbf{0.375}}$

D= 4.07 ft., use 4.0" pipe.

For 48" pipe, full flow capacity $Q=\dfrac{0.46\ D^{2.67}S^{0.5}}{n} = \dfrac{0.46\ (4.0)^{2.67}*(0.005)^{0.5}}{0.013} = 101.3$ cfs
since full flow capacity is less than design Q, velocity is Q/A.

Design Velocity = 107.7/12.566=8.57 ft/sec. = 514 ft/min.

Travel time = L/V = (140/514) =0.27 min.

Pipe 1 Length=100, slope 0.005
Area "A" Additional CA= 3.30 Ac.
Total CA= 30.85+3.30=34.15

Inlet Tc= 10 min., system time = 18.87+0.27= 19.14, use system Tc of 19.14 minutes.
I= 3.43 in/hr.
Q=34.15*3.43=117.2 cfs.

Minimum pipe diameter for full flow:
$D=1.33[Qn/S^{\mathbf{0.5}}]^{\mathbf{0.375}}$

$D=1.33[117.2*0.013/(0.005)^{\mathbf{0.5}}]^{\mathbf{0.375}}$

D= 4.21 ft. Use 4.0 pipe.

For 48" pipe, full flow capacity $Q=\dfrac{0.46\ D^{2.67}S^{0.5}}{n} = \dfrac{0.46\ (4.0)^{2.67}*(0.005)^{0.5}}{0.013} = 101.3$ cfs

since full flow capacity is less than design Q, velocity is Q/A.

Appendix 13.A Example Problem (continued)

Step 2. Determine the Design Discharge and initial pipe sizes. (continued)

Pipe 1 (continued)
Design Velocity = 107.7/12.566=8.57 ft/sec. = 514 ft/min.

Travel time = L/V = (100/514) =0.19 min.

In the next section junction losses will be computed based on the assumed pipe sizes and full flow velocities.

Junction Losses

Step 3. Determine the junction loss coefficients for the initial system design.

At station 101+00; Upstream end of Pipe 1

Connector pipe at a “T”.

$$H=\frac{Q_oV_o-Q_iV_i-Q_lV_l\cos(\theta)}{0.5(A_o+A_i)g}+\frac{V_i^2-V_o^2}{2g}$$

Inlet Pipe 2, Q=107.70
Outlet Pipe 1, Q=117.18
Connector Pipe “A”, Q=9.48
$\theta = 90°$

$$H=\frac{117.18*9.32-107.7*8.57-9.48*5.36*\cos(90°)}{0.5(12.57+12.57)*32.2}+\frac{8.57^2-9.32^2}{64.4}=0.21\text{ ft}$$

$K= H/(V_0^2/2g) = 0.21/(9.32^2/32.2)=0.155$

Pipe	Q	Diameter	A	V	$V^2/2g$	Theta	Hj, ft	K
Inlet Pipe 2	107.70	48	12.57	8.57	1.141	180°	0.21	0.155
Pipe “A”	9.48	18	1.77	5.36	0.447	90°		
Outlet Pipe 1	117.18	48	12.57	9.32	1.350	0°		

At Station 102+40; 2 connector pipes at a manhole

Loss co-efficient, $K= K_0\, C_D\, C_d\, C_Q\, C_B\, C_P$
Where K_0= initial head loss coefficient based on relative access hole size.
C_D= Correction for pipe diameter.
C_d= Correction for flow depth.

Appendix 13.A Example Problem (continued)

At Station 102+40; 2 connector pipes at a manhole (continued)

C_Q= Correction for relative flow.
C_B= Correction for benching.
C_P= Correction for plunging flow.

K_0= initial head loss coefficient

K_0=0.1 (b/Do)(1-sin(□)+1.4 (b/Do)$^{0.15}$ sin (□)

For Pipe 3, □=180□
K_0= 0.1(48/48)(1-sin(180□))+1.4(48/48)$^{0.15}$sin(180□)
= 0.1(1)(1-(0))+1.4(1)$^{0.15}$(0)=0.1

For Pipe “B”, □=90□
K_0= 0.1(48/48)(1-sin(90□))+1.4(48/48)$^{0.15}$sin(90□)
= 0.1(1)(1-1))+1.4(1)0.15(1)=0+1.4 = 1.4

For Pipe “C”, □=90□
= 0.1(48/48)(1-sin(90□))+1.4(48/48)$^{0.15}$sin(90□)
= 0.1(1)(1-1))+1.4(1)0.15(1)=0+1.4 = 1.4

C_D= Correction for pipe diameter.

Since this system is initially designed for flow depth approximately equal to the diameter of the pipe, d/Do=1, then C_D= 1.

C_d= Correction for flow depth.

C_d= 0.5(d/Do)$^{0.6,}$ since this system is initially designed for flow depth approximately equal to the diameter of the pipe, d/Do=1, then C_d = 0.5

C_Q= Correction for relative flow.

C_Q= 1+(1-2*sin(□)*(1-Q_i/Q_o)$^{0.75}$

Inlet Pipe 3, Q=91.29 cfs, □= 180□
Connector Pipe “B”, Q=9.01 cfs, □=90□
Connector Pipe “C”, Q=7.40 cfs, □=90□
Outlet Pipe 21, Q=107.7

For the inlet pipe 3,
C_Q= 1+(1-2*sin(180□))*(1-(91.29/107.7)$^{0.75}$)
C_Q= 1+(1-0)*(1-(0.848) $^{0.75}$) = 1+(1*0.116) =1.116

Appendix 13.A Example Problem (continued)

At Station 102+40; 2 connector pipes at a manhole (continued)

For connector pipe "B", Q=9.01 cfs, θ=90
C_Q= 1+(1-2*sin(90))*(1-(9.01/107.7)$^{0.75}$)
C_Q= 1+(1-2)*(1-(0.084) $^{0.75}$) = 1+(-1*0.844) =0.155

For connector pipe "C", Q=7.4 cfs, θ=90°
C_Q= 1+(1-2*sin(90°))*(1-(7.40/107.7)$^{0.75}$)
C_Q= 1+(1-2)*(1-(0.069) $^{0.75}$) = 1+(-1*0.865) =0.135

C_Q= Correction for relative flow.

Pipe	Q	Diameter	A	V	$V^2/2g$	Theta	C_Q
Inlet Pipe 3	91.29	48	12.57	7.26	0.819	180	1.116
Pipe "B"	9.01	24	3.14	2.87	0.128	90	0.155
Pipe "C"	7.40	24	3.14	2.23	0.086	90	0.135
Outlet Pipe 4	107.7	48	12.57	1.141	1.141	0	

C_B= Correction for benching.

Since this system is initially designed for flow depth approximately equal to the diameter of the pipe, d/D_O=1, then for a half bench C_B= 0.15

C_P= Correction for plunging flow.

$$C_P= 1+0.2\left[\frac{h}{D_o}\right]*\left[\frac{(h-d)}{D_o}\right]$$

For this location, the connector pipes inverts are within the diameter of the outflow pipe, therefore C_P= 1.

At Station 102+40;
In summary, $K= K_0\ C_D\ C_d\ C_Q\ C_B\ C_P$

Pipe	K_0	C_D	C_d	C_Q	C_B	C_P	K	Vel. Hd	H_f, ft.
3	0.1	1	0.5	1.116	0.15	1	0.0084	1.14	0.01
"B"	1.4	1	0.5	0.155	0.15	1	0.016	1.14	0.02
"C"	1.4	1	0.5	0.135	0.15	1	0.014	1.14	0.02

At station 103+70; connector pipe at a "Y", 45 degrees with a sudden change in pipe size from 42 inch to 48 inch.

Appendix 13.A Example Problem (continued)

At Station 103+70; (continued)

For junction:

$$H_j=\frac{Q_oV_o-Q_iV_i-Q_lV_l\cos(□)}{0.5(A_o+A_i)g}+\frac{V_i^2-V_o^2}{2g}$$

Inlet Pipe 4, Q=84.32
Outlet Pipe 3, Q=91.29
Connector Pipe "D", Q=6.97, □ = 135□

$$H=\frac{91.29*7.26-84.32*6.71-6.97*2.22*\cos(45□)}{0.5(9.62+12.57)*32.2}+\frac{(6.21^2-7.26^2)}{64.4} = 0.093 \text{ ft.}$$

$K= H/(V_0^2/2g) = 0.093/(7.26^2/32.2)=0.114$

Pipe	Q	Diameter	A	V	$V^2/2g$	Theta	Hj, ft	K
Inlet Pipe 4	84.32	48	12.57	6.71	0.699	180□	0.093	0.114
Pipe "D"	6.97	24	3.14	2.22	0.076	135□		
Outlet Pipe 3	91.29	48	12.57	7.26	0.819	0		

For sudden expansion:

$He = K_e(Vi^2/2g)$

For Do/Di=(48/42)=1.14 and Vi=8.76 ft/sec, K_e = 0.1

$He = 0.1[(8.76)^2/64.4] = 0.119$

Therefore loss at upstream end for 42-inch pipe is 0.093+0.119=0.212 ft,

$K=0.212/(7.26^2/64.4)= 0.26$

At Station 105+00: 3 connector pipes at a manhole

Loss co-efficient, $K= K_0\ C_D\ C_d\ C_Q\ C_B\ C_P$

Where K_0= initial head loss coefficient based on relative access hole size.

C_D= Correction for pipe diameter.
C_d= Correction for flow depth.
C_Q= Correction for relative flow.
C_B= Correction for benching.
C_P= Correction for plunging flow.

Appendix 13.A Example Problem (continued)

At Station 105+00: 3 connector pipes at a manhole (continued)

K_0= initial head loss coefficient

Pipe 5, □=180□

= $0.1(48/42)(1\text{-}\sin(180□))+1.4(48/42)^{0.15}\sin(180□)$

= $0.1(1.14)(1\text{-}(0))+1.4(114)^{0.15}(0) = 0.114$

Connector Pipe 6, Q=35.39, □=135□

= $0.1(48/42)(1\text{-}\sin(135□))+1.4(48/42)^{0.15}\sin(135□)$

= $0.1(1.14)(1\text{-}(0.707))+1.4(1.14)^{0.15}(0.707) = 0.033+1.01 = 1.04$

Connector Pipe “E”, Q=20.17, □=90□

= $0.1(48/42)(1\text{-}\sin(90□))+1.4(48/42)^{0.15}\sin(90□)$

= $0.1(1.14)(1\text{-}(1))+1.4(1.14)^{0.15}(1) = 1.43$

C_D= Correction for pipe diameter.

Since this system is initially designed for flow depth approximately equal to the diameter of the pipe, d/Do=1, then C_D= 1.

C_d= Correction for flow depth.

$C_d= 0.5(d/Do)^{0.6,}$ since this system is initially designed for flow depth approximately equal to the diameter of the pipe, d/Do=1, then $C_d = 0.5$

C_Q= Correction for relative flow.

$C_Q= 1+(1\text{-}2*\sin(□)*(1\text{-}Q_i/Q_o)^{0.75}$

Inlet Pipe 5, Q=28.76, □=180□
Outlet Pipe 4, Q=84.32
Connector Pipe 6, Q=35.39, □=135□
Connector Pipe “E”, Q=20.17, □=90□

For the inlet pipe 5,

$C_Q= 1+(1\text{-}2*\sin(180□))*(1\text{-}(28.76/84.32))^{0.75}$

$C_Q= 1+(1\text{-}0)*(1\text{-}0.341)^{0.75} = 1+(1)*(0.731) =1.731$

For connector pipe 6, Q=35.39 cfs, □=135□

$C_Q= 1+(1\text{-}2*\sin(135□))*(1\text{-}(35.39/84.32))^{0.75}$

$C_Q= 1+(1\text{-}2(0.707))*(1\text{-}0.420)^{0.75} = 1+(\text{-}0.414)*(0.58) =0.725$

Appendix 13.A Example Problem (continued)

At Station 105+00: 3 connector pipes at a manhole (continued)

For connector pipe “E”, Q=20.17 cfs, □=90□

C_Q= 1+(1-2*sin(90□))*(1-(20.17/84.32))$^{0.75}$

C_Q= 1+(1-2)*(1-0.239) $^{0.75}$ = 1+(-1)*(0.815) =0.185

Pipe	Q	Diameter	A	V	$V^2/2g$	Theta	C_Q
Inlet Pipe 5	28.76	36	7.07	4.07	0.257	180□	1.731
Inlet Pipe 6	35.39	30	4.91	7.21	0.807	135□	0.725
Pipe “E”	20.17	30	4.91	4.11	0.262	90□	0.185
Outlet Pipe 4	84.32	42	9.62	8.76	1.193	0□	

C_B= Correction for benching.

Since this system is initially designed for flow depth approximately equal to the diameter of the pipe, d/Do=1, then for half bench C_B= 0.15

C_P= Correction for plunging flow.

$$C_P = 1+0.2\left[\frac{h}{D_o}\right]*\left[\frac{(h-d)}{D_o}\right]$$

For this location, the connector pipes inverts are within the diameter of the outflow pipe, therefore C_P= 1.

At Station 105+00:

In summary, K= K_0 C_D C_d C_Q C_B C_P

Pipe	K_0	C_D	C_d	C_Q	C_B	C_P	K	Vel. Hd	H_f, ft.
5	0.114	1	0.5	1.731	0.15	1	0.015	1.19	0.02
6	1.04	1	0.5	0.725	0.15	1	0.0565	1.19	0.07
“E”	1.43	1	0.5	0.185	0.15	1	0.016	1.19	0.02

At Station 107+75; Manhole with 180 degree connection.

K_0= initial head loss coefficient

Pipe “5”, Q=28.76, □=180□

= 0.1(48/36)(1-sin(180□))+1.4(48/36)$^{0.15}$sin(180□)

= 0.1(1.33)(1-(0))+1.4(1.33)$^{0.15}$(0) = 0.133

Appendix 13.A Example Problem (continued)

At Station 107+75; Manhole with 180 degree connection. (continued)

$C_D = 1$
$C_d = 0.5$
$C_Q = 1$
$C_B = 0.15$
$C_P = 1$

In summary, $K = K_0\ C_D\ C_d\ C_Q\ C_B\ C_P$

$$K = 0.133*(1)*(0.5)*(1)*(0.15)*(1) = 0.01$$

At Station 108+05; Open pipe inlet.

Ke=0.2, concrete pipe with headwall.

At Station B106+25; Manhole with connector pipe.

Inlet Pipe 7, Q=21.82
Outlet Pipe 6, Q=35.39
Connector Pipe “G”, Q=13.57

Loss co-efficient, $K = K_0\ C_D\ C_d\ C_Q\ C_B\ C_P$

Where K_0= initial head loss coefficient based on relative access hole size.
C_D= Correction for pipe diameter.
C_d= Correction for flow depth.
C_Q= Correction for relative flow.
C_B= Correction for benching.
C_P= Correction for plunging flow.

K_0= initial head loss coefficient

b=48, Do=30”

Pipe 7, □=180□

$K_0 = 0.1(48/30)(1-\sin(180□))+1.4(48/30)^{0.15}(\sin(180□))$
$= 0.1(1.6)(1-(0))+1.4(1.6)^{0.15}(0) = 0.16$

Connector Pipe “G”, □ = 90□

$K_0 = 0.1(48/30)(1-\sin(90□))+1.4(48/30)^{0.15}(\sin(90□))$
$= 0.1(1.6)(1-(1))+1.4(1.6)^{0.15}(1) = 1.50$

Appendix 13.A Example Problem (continued)

At Station B106+25; Manhole with connector pipe. (continued)

C_D= Correction for pipe diameter.

Since this system is initially designed for flow depth approximately equal to the diameter of the pipe, d/Do=1, then C_D= 1.

C_d= Correction for flow depth.

$C_d= 0.5(d/D_o)^{0.6}$, since this system is initially designed for flow depth approximately equal to the diameter of the pipe, d/D_o=1, then C_d = 0.5

C_Q= Correction for relative flow.

$C_Q= 1+(1-2*\sin(□)*(1-Q_i/Q_o)^{0.75}$

Inlet Pipe 7, Q=21.82, □= 180□
Connector Pipe "G", Q=13.57, □ = 90□
Outlet Pipe 6, Q=35.39,

For the inlet pipe 7,
$C_Q= 1+(1-2*\sin(180□))*(1-(21.82/35.39))^{0.75}$
$C_Q= 1+(1-0)*(1-0.617)^{0.75} = 1+(1)*(0.487) =1.487$

For connector pipe "G", Q=13.57 cfs, □=90□
$C_Q= 1+(1-2*\sin(90□))*(1-(13.57/35.39))^{0.75}$
$C_Q= 1+(1-2)*(1-0.383)^{0.75} = 1+(-(1)*(0.696) =0.304$

Pipe	Q	Diameter	A	V	$V^2/2g$	Theta	C_Q
Inlet Pipe 7	21.82	24	3.14	6.95	0.749	180□	1.487
Pipe "F"	13.57	24	3.14	4.32	0.280	90□	0.304
Outlet Pipe 6	35.39	30	4.91	7.21	0.807	0□	

C_B= Correction for benching.

Since this system is initially designed for flow depth approximately equal to the diameter of the pipe, d/Do=1, then for half bench C_B= 0.15

C_P= Correction for plunging flow.

$$C_P= 1+0.2\left[\frac{h}{D_o}\right]*\left[\frac{(h-d)}{D_o}\right]$$

For this location, the connector pipes inverts are within the diameter of the outflow pipe, therefore C_P= 1.

Appendix 13.A Example Problem (continued)

Step 3. Determine the junction loss coefficients for the initial system design. (continued)

At Station B106+25;
In summary, $K = K_0\ C_D\ C_d\ C_Q\ C_B\ C_P$

Pipe	K_0	C_D	C_d	C_Q	C_B	C_P	K	Vel. Hd	H_f, ft.
7	0.16	1	0.5	1.487	0.15	1	0.018	0.81	0.01
“F	1.50	1	0.5	0.304	0.15	1	0.004	0.81	0.00

At Station B108+25, connector pipe at 135 degrees.

K_0= initial head loss coefficient

Pipe “7”, Q=21.82, □=135□
= 0.1(48/24)(1-sin(135□))+1.4(48/24)$^{0.15}$sin(135□)
= 0.1(2.0)(1-(0.707))+1.4(2.0)$^{0.15}$(0.707) = 0.0.594 + 1.0983 = 1.158

At Station B108+25, connector pipe at 135 degrees. (continued)

$C_D = 1$
$C_d = 0.5$
$C_Q = 1$
$C_B = 0.15$
$C_P = 1$

In summary, $K = K_0\ C_D\ C_d\ C_Q\ C_B\ C_P$

K= 1.158*(1)*(0.5)*(1)*(0.15)*(1) = 0.087

At Station B108+55; Drop inlet.

Ke=0.2, concrete pipe with headwall.

In the next section based on the junction losses determined above and the actual velocity heads, the hydraulic grade line will be computed

Appendix 13.A Example Problem (continued)

Step 4. Determine the hydraulic grade line for the initial system design.

Hydraulic Grade Line Computation

Flow at outfall is free flow, set HGL at crown of pipe.

Pipe 1. Station 100+00 to Station 101+00

Downstream invert = 101.00.
Upstream invert = 101.50

Downstream Crown=101.00+4.0=105.00

Set HGL at pipe crown, HGL_1=105.00

Pipe	Length	Q	Diameter	A	Vel	H_{vel}	S_f, %	H_f
1	100	117.2	4.0	12.57	9.33	1.35	0.6761	0.68

EGL_1 = HGL_1+ H_{vel}
= 105.00+1.35 = 106.35

For upstream end of pipe, check if flow is open channel or pressure.
EGL_2 =EGL_1+H_f
= 106.35+0.68 =
= 107.03

HGL_2 at U.S. =EGL_2 - H_{vel}
HGL_2 = 107.03-1.35 = 105.68
Crown at U.S.= 105.50, therefore pressure flow.

Pipe 2: Station 101+00 to Station 102+40
Downstream invert = 101.5
Upstream invert = 102.20

At D.S. of pipe 2,
At junction, station 101+00, K=0.155
Junction loss = 0.155*1.35=0.21 ft.

EGL_1= EGL_2 from pipe 1+ junction loss= 107.03+0.21 = 107.24

HGL_1=107.24 – 1.14= 106.10
Crown at D.S.= 101.5+4.0 = 105.50, therefore pressure flow.

Pipe	Length	Q	Diameter	A	Vel	H_{vel}	S_f, %	H_f
2	140	107.7	4.0	12.57	8.57	1.14	0.5645	0.79

Appendix 13.A Example Problem (continued)

Pipe 2: Station 101+00 to Station 102+40 (continued)

For upstream end of pipe, check if flow is open channel or pressure.

EGL_2 = EGL_1+H_f
= 107.24 + 0.79
= 108.03

HGL_2 at U.S. =EGL_2 - H_{vel}
= 108.03-1.14 = 106.89
Crown at U.S.= 106.2, therefore pressure flow.

Pipe 3: Station 102+40 to Station 103+70
Downstream invert = 102.20
Upstream invert = 103.11
At D.S. of pipe 3.
At junction, station 102+40,
For pipe 3, K=0.0084
Junction loss = 0.01 ft.

EGL_1= EGL_2 from pipe 1+ junction loss= 108.03+0.01 = 108.04

HGL_1=108.04 – 0.82= 107.22
Downstream Crown=106.20, pressure flow

Pipe	Length	Q	Diameter	A	Vel	H_{vel}	S_f, %	H_f
3	130	91.29	4.0	12.57	7.26	0.82	0.4056	0.53

For upstream end of pipe, check if flow is open channel or pressure.

EGL_2 = EGL_1+H_f
= 108.04 + 0.53
= 108.57
HGL_2 at U.S. =EGL_2 - H_{vel}
= 108.57-0.82 = 107.75

Crown at U.S.= 107.11, therefore pressure flow.

Appendix 13.A Example Problem (continued)

Step 4. Determine the hydraulic grade line for the initial system design. (continued)

Pipe 4: Station 103+70 to 105+00
Downstream invert = 103.61
Upstream invert = 105.43

At D.S. of pipe 4.
At junction, station 103+70
For pipe 4, K=0.26
Junction loss = 0.26*0.82 = 0.21 ft.

EGL_1= EGL_2 from pipe 3+ junction loss= 108.57+0.21 = 108.78

HGL_1=108.78 – 1.19= 107.59
Downstream Crown=103.61+3.50 = 107.11, therefore pressure flow.

Pipe	Length	Q	Diameter	A	Vel	H_{vel}	S_f, %	H_f
4	130	84.32	3.5	9.62	8.76	1.19	0.7052	0.92

For upstream end of pipe, check if flow is open channel or pressure.
EGL_2 = EGL_1+H_f
= 108.78+ 0.92
= 109.70

HGL_2 at U.S. =EGL_2 - H_{vel}
= 109.70-1.19 = 108.51

Crown at U.S.= Invert + Dia.= 105.43 + 3.50 = 108.93, therefore open channel flow.

At a pipe slope of 0.01 flow is supercritical. From critical depth chart for circular pipe in chapter 9, critical depth=2.86 ft.
D/D=2.86/3.5= 0.817
From Table 13-B.2 for a flow depth/Diameter ratio of 0.817, C=0.687
A=0.687(3.5^2)=8.42 sq.ft., Vel=Q/A = 84.32/8.42 = 10.01 ft/sec.

Check with Figure 13-3, for a depth of 2.86 ft, d/D=0.82, V/V_{full}= 1.15.
V_c=1.15*8.76=10.07 ft/sec.

Use critical depth of 2.86 ft., V_n=10.01 ft/sec, Vel.Hd=1.56 ft.

HGL_2 = Invert+d_c = 105.43+2.86 = 108.29 > 107.42 at D.S.,
EGL_2=HGL_2 + H_{vel} = 108.29 + 1.56 = 109.85

Appendix 13.A Example Problem (continued)

Step 4. Determine the hydraulic grade line for the initial system design. (continued)

Pipe 5: Station 105+00 to Station 107+80
Downstream invert = 105.93
Upstream invert = 109.99

At D.S. of pipe 5

At junction, station 105+00
Pipe 5 K=0.015
Junction loss = 0.015*1.56 = 0.02 ft.

EGL_1= EGL_2 from pipe 4+ junction loss= 109.85+0.02 = 109.87
For full flow, Hv = 0.26 ft.

HGL_1=109.87 – 0.26= 109.61

Crown = 105.93 + 3.0 = 108.93< 109.61, therefore pressure flow.

Pipe	Length	Q	Diameter	A	Vel	H_{vel}	S_f, %	H_f
5	280	28.76	3.0	7.07	4.07	0.26	0.1885	0.53

For upstream end of pipe, check if flow is open channel or pressure.
EGL_2 = EGL_1+H_f
= 109.87+ 0.53
= 110.40

HGL_2 at U.S. =EGL_2 - H_{vel}
= 110.40-0.26 = 110.14

At U.S. of pipe 5
Invert = 109.99
Crown = 109.99 + 3.0 = 112.99, therefore open channel flow.

For a 36-inch pipe at a slope of 0.0145 and a discharge of 28.76 cfs, flow is supercritical.
At upper end of pipe, flow depth is critical depth.
d_c=1.73 ft and d_n = 1.24 ft.

From critical depth chart for circular pipe in chapter 9, critical depth=1.73 ft.
From Table 13-B.2, for a depth of 1.73 ft., d/D=0.577, C=0.469
A=0.469*(3.0^2) = 4.22
V = 28.76/4.22 = 6.82 ft/sec.

Appendix 13.A Example Problem (continued)

Step 4. Determine the hydraulic grade line for the initial system design. (continued)

Pipe 5: Station 105+00 to Station 107+80 (continued)

H_{vel} = 0.72 ft.

HGL_2 = Invert+d_c = 109.99 + 1.73 = 111.72 > 109.61 at D.S.,

EGL_2=HGL_2 + H_{vel} = 111.72 + 0.72 = 112.44

Pipe 13: Station 107+80 to Station 107+90

Downstream invert = 109.99
Upstream invert = 110.09

At D.S. of pipe 13

At junction, station 107+75
Pipe 13 K=0.01
Junction loss = 0.01*0.72 = 0.01 ft.

EGL_1= EGL_2 from pipe 5+ junction loss= 112.44+0.01 = 112.45

Crown = 109.99 + 3.0 = 112.99>112.45. Therefore open channel flow.

For slope of 0.01, flow is supercritical.
For a 36-inch pipe at a slope of 0.01 and a discharge of 28.76 cfs, d_c=1.73 ft and d_n = 1.24 ft.

$$HGL_1 = \frac{(dc+D)}{2} + \text{invert}$$

$$= \frac{(1.73+3.0)}{2} + 109.99 = 112.36 > 111.79$$, downstream water surface does not control.

d = 112.36-109.99 =2.37 > 1.73 depth greater than critical depth.
d/D = 2.37/3.0 = 0.79
From table 13-B.2, C= 0.666
A=C*D^2 = 0.666*(3^2) = 6.000
V = 28.76/6.000 = 4.79 ft/sec., H_{vel} = 0.36

EGL_1=HGL_1 + H_{vel} = 112.36 + 0.36 = 112.72

Pipe	Length	Slope, ft/ft	Q	Diameter	A	Vel, full
13	10	0.01	28.76	3.0	7.07	4.07

Check flow depth at upstream end of pipe.

At U.S. of pipe 13
Invert = 110.09
Crown = 110.09 + 3.0 = 113.09

Appendix 13.A Example Problem (continued)

Pipe 13: Station 107+80 to Station 107+90 (continued)
From critical depth chart for circular pipe in chapter 9, critical depth=1.73 ft.
HGL_2 = Invert+d_c = 110.09+1.73 = 111.82 < 112.36 at D.S.
Therefore depth at entrance = 112.36-110.09 = 2.27 ft.
From Table 13-B.2, for a depth of 2.27 ft., d/D=0.757, C= 0.638

A=0.638*(3.0^2) = 5.742
V = 28.76/5.742 = 5.01 ft/sec.
H_{vel} = 0.39 ft.

EGL_2=HGL_2 + H_{vel} = 112.36 + 0.39 = 112.75

This is a culvert headwall with a supercritical slope, check using culvert analysis.
Results are a headwater at entrance equal to 112.69. This is in outlet control, influenced by "tailwater" condition.

Pipe 6: Station 105+00 to Station B106+25

Downstream invert = 106.43
Upstream invert = 107.68

At D.S. of pipe 6
Crown = 106.43 + 2.5 = 108.93

At junction, station 105+00
Pipe 6 K=0.014
Junction loss = 0.014*0.30 = 0.004 ft., say 0.00

EGL_1= EGL_2 from pipe 4+ junction loss= 109.92+0.00 = 109.92
For full flow, Hv = 0.81 ft.

HGL_1=109.92 – 0.81= 109.11

Crown = 106.43 + 2.50 = 108.90 < 109.11, therefore pressure flow.

Pipe	Length	Q	Diameter	A	Vel	H_{vel}	S_f, %	H_f
6	125	35.39	2.5	4.91	7.21	0.81	0.6652	0.83

Appendix 13.A Example Problem (continued)

Pipe 6: Station 105+00 to Station B106+25 (continued)

For upstream end of pipe, check if flow is open channel or pressure.
For pressure flow,
$EGL_2 = EGL_1 + H_f$
$= 109.92 + 0.83$
$= 110.75$

HGL_2 at U.S. $= EGL_2 - H_{vel}$
$= 110.68 - 0.81 = 109.87$

At U.S. of pipe 6
Invert = 107.68
Crown = 107.68 + 2.5 = 110.18 > 109.87, therefore open channel flow.

At a friction slope of 0.006652 and a pipe slope of 0.01, the flow becomes open channel at

$$L = \frac{(109.11-108.93)}{(0.0100 - 0.006652)} = \frac{0.18}{0.003348} = 53.8 \text{ ft.}$$

For slope of 0.010, flow is supercritical.

For a 30-inch pipe at a slope of 0.01 and a discharge of 35.39 cfs, d_c=2.02 ft and d_n = 1.79 ft.
At upstream end, invert + dc = 107.68 + 2.02 = 109.70 > 109.11, therefore depth is controlled by critical depth at upstream end.

From Table 13-B.2, for a depth of 2.02 ft., d/D=0.808, C=0.680
$A=0.680*(2.5^2) = 4.25$ sq.ft.
V = 35.39/4.25 = 8.33 ft/sec.
H_{vel} = 1.08 ft.

$HGL_2 = \text{Invert}+d_c = 107.68+2.02 = 109.70 > 109.11$ at D.S.,
$EGL_2=HGL_2 + H_{vel} = 109.70 + 1.08 = 110.78$

Pipe 7: Station B106+25 to Station B108+75
Downstream invert = 108.25
Upstream invert = 111.75

At D.S. of pipe 7
Invert = 108.18
Crown = 108.18+ 2.0 = 110.18

Appendix 13.A Example Problem (continued)

Pipe 7: Station B106+25 to Station B108+75 (continued)
At junction, station B106+25
Pipe 7 K=0.02
Junction loss = 0.02*1.07 = 0.02 ft.

Check downstream control,
HGL_1=109.70+0.02=109.72 < 110.18, therefore open channel flow.

Pipe	Length	Slope, ft/ft	Q	Diameter	A	Vel, full
7	250	0.0140	21.82	2.0	3.14	6.95

For slope of 0.014, flow is supercritical.
For a 24-inch pipe at a slope of 0.014 and a discharge of 21.82 cfs, d_c=1.67 ft and d_n = 1.37 ft.

d = 109.72 - 108.18 = 1.54 ft. < d_c.
d/D = 1.54/2.0 = 0.77
From table 13-B.2, C= 0.649. A=C*D^2 = 0.649*(2^2) = 2.596
V = 21.82/2.596 = 8.41 ft/sec.

Check using Figure 13-3, A/A_f=0.84, V/V_f=1.15
A = 0.83*3.14 = 2.61, V=21.82/2.61 = 8.36 ft/sec.
Check V/V_f=1.15*6.95 = 7.99 ft/sec

Use velocity = 8.41 ft/sec, H_{vel} = 1.10
EGL_1=HGL_1 + H_{vel} = 109.72 + 1.10 = 110.82

At U.S. of pipe 7, station B108+75
Crown = 111.68 + 2.00 = 113.68
For slope of 0.015, flow is supercritical.
At upper end of pipe, flow depth is critical depth, d_c=1.67 ft.

From Table 13-B.2, for a depth of 1.67 ft., d/D=0.835, C=0.700
A=0.700*(2.0^2) = 2.80
V = 21.82/2.80 = 7.79 ft/sec.
H_{vel} = 0.94 ft.
HGL_2 = Invert+d_c = 111.68+1.67 = 113.35 > 109.72 at D.S.,
EGL_2=HGL_2 + H_{vel} = 113.35 + 0.94 = 114.29

Appendix 13.A Example Problem (continued)

Step 4. Determine the hydraulic grade line for the initial system design. (continued)

Pipe 15: Station B108+75 to Station B109+05

Downstream invert = 111.68
Upstream invert = 112.13

At D.S. of pipe 15

Pipe is at 135
At junction, station B108+75, K=0.09
Junction loss = 0.09*0.94 = 0.08 ft.

HGL_1=113.35+0.08=113.43
Crown = 111.68+2.0 = 113.68 > 113.43. Therefore open channel flow.

Pipe	Length	Slope, ft/ft	Q	Diameter	A	Vel, full
15	30	0.015	21.82	2.0	3.14	6.95

Depth of flow, 113.43-111.68 = 1.75 ft.

For a 24-inch pipe at a slope of 0.015 and a discharge of 21.82 cfs, d_c=1.67 ft and d_n = 1.34 ft.

D/D = 1.75/2.0 = 0.875, from Table 13-B.2 C= 0.729
A= 0.729 (2^2)= 2.92 sq. ft.
V = 21.82/2.92 = 7.47 ft/sec., H_{vel} = 0.87
EGL_1=HGL_1 + H_{vel} = 113.43+ 0.87 = 114.30

upstream end check depth of flow: 1) HGL is Invert + d_c.
2.) HGL at D/S end

HGL_2= 112.13 + 1.67 = 113.80 > downstream water surface of 113.43, use water depth of 1.67 ft.
For a depth of 1.67 ft, d/D = 0.835, C= 0.7005
A= 0.7005 (2^2)=2.80
V = 21.82/2.80 = 7.79 ft/sec., H_{vel} = 0.94
EGL_2=HGL_2 + H_{vel} = 113.80+ 0.94 = 114.74

This is a drop inlet with a supercritical slope, check using culvert analysis.
Results are a headwater at entrance equal to 115.03. This pipe is in inlet control. If Gutter elevation is 116.2, then “freeboard” is 1.10 ft, greater than 0.5 ft. Outlet pipe does not affect inflow through inlet.

Appendix 13.A Example Problem (continued)

Hydraulic Grade Line Computation Form.

Station	TW HGL_o	D_0	Q_o	L_o	V_o	$V_o^2/2g$	H_o	S_{fo}	H_f	K_o	C_D	C_d	C_Q	C_p	C_B	K	$K(V_o^2/2g)$	EGL_0 2+7	EGL_i 10+18 +19	HGL_i EGL_i-7	Crown Elev
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)
100+00 Pipe 1	105.00	4.0	117.2	100.0	9.33	1.35	1.35	0.6761	0.68	-	-	-	-	-	-	0.155	0.21	106.35	107.03	105.68	105.50
101+00 Pipe 2	106.10	4	107.7	140.0	8.57	1.14	0.21	0.5645	0.79	0.1	1	0.5	1.116	0.15	1	0.0084	0.01	107.24	108.03	106.89	106.20
102+40 Pipe 3	107.22	4	91.29	130.0	7.26	0.82	0.01	0.4056	0.53	0.114	-	-	-	-	-	0.114	0.21	108.04	108.57	107.75	107.11
103+70 Pipe 4	107.59	3.5	84.32	130.0	8.76	1.19	0.093	0.7052	0.92	0.26	1	0.5	-	0.15	1	0.26	0.02	108.78	109.85*	108.29*	108.93
105+00 Pipe 5	109.61	3	28.76	280.0	4.07	0.26		0.1885	0.53	0.1	1	0.5	1.731	0.15	1	0.0087	0.01	109.87	112.44*	111.72*	112.99
107+80 Pipe 13	112.36	3.0	28.76	10.0	4.79	0.36		0.01		0.133	1	0.5	1	1	0.15	0.01	0.01	112.72*	112.75*	112.36*	113.09
B105+00 Pipe 6	109.11	2.5	35.39	125.0	7.21	0.81		0.6652	0.83	1.04	1	0.5	0.725	0.15	1	0.14	0.004	109.92	110.78*	109.70*	110.18
B106+25 Pipe 7	109.72	2.0	21.82	250.0	8.41	1.10				0.16	1	0.5	1.487	0.15	1	0.018	0.01	110.82	114.29*	113.35*	113.75
B108+75 Pipe 15	113.42	3.0	21.82	30	7.47	0.87										0.09	0.08	114.30	114.74*	113.80*	114.13

Appendix 13.B Geometric Properties of Circular Channels

Table 13-B.1

Geometric Properties of circular open channels

Area(A)	$\frac{(\text{depth})^2(\square-\sin(\square))}{8}$
Wetted Perimeter (P)	(depth)(□/2)
Hydraulic Radius (R)	$\frac{(\text{depth})*(1-\sin(\square)/(\square))}{4}$
Top Width (T)	(depth)sin(□/2)
Hydraulic Depth (D_h)	$\frac{(\text{depth})*(\square-\sin(\square))}{8*(\sin(\square/2))}$

□ = angle of subtended arc, in radians.

Table 13-B.2

Coefficient for Area of flow in circular conduit flowing part full.

depth/D	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
0.0	0.0000	0.0013	0.0037	0.0069	0.0105	0.0147	0.0192	0.0242	0.0294	0.0350
0.1	0.0409	0.0470	0.0534	0.0600	0.0668	0.0739	0.0811	0.0885	0.961	0.1039
0.2	0.1118	0.1199	0.1281	0.1365	0.1449	0.1535	0.1623	0.1711	0.1800	0.1890
0.3	0.1982	0.2074	0.2167	0.2260	0.2355	0.2450	0.2546	0.2642	0.2739	0.2836
0.4	0.2934	0.3032	0.3130	0.3229	0.3328	0.3428	0.3527	0.3627	0.3727	0.3827
0.5	0.393	0.403	0.413	0.423	0.433	0.443	0.453	0.462	0.472	0.482
0.6	0.492	0.502	0.512	0.521	0.531	0.540	0.550	0.559	0.569	0.578
0.7	0.587	0.596	0.605	0.614	0.623	0.632	0.640	0.649	0.657	0.666
0.8	0.674	0.681	0.689	0.697	0.704	0.712	0.719	0.725	0.732	0.738
0.9	0.745	0.750	0.756	0.761	0.766	0.771	0.775	0.779	0.782	0.784

Appendix 13.B Geometric Properties of Circular Channels

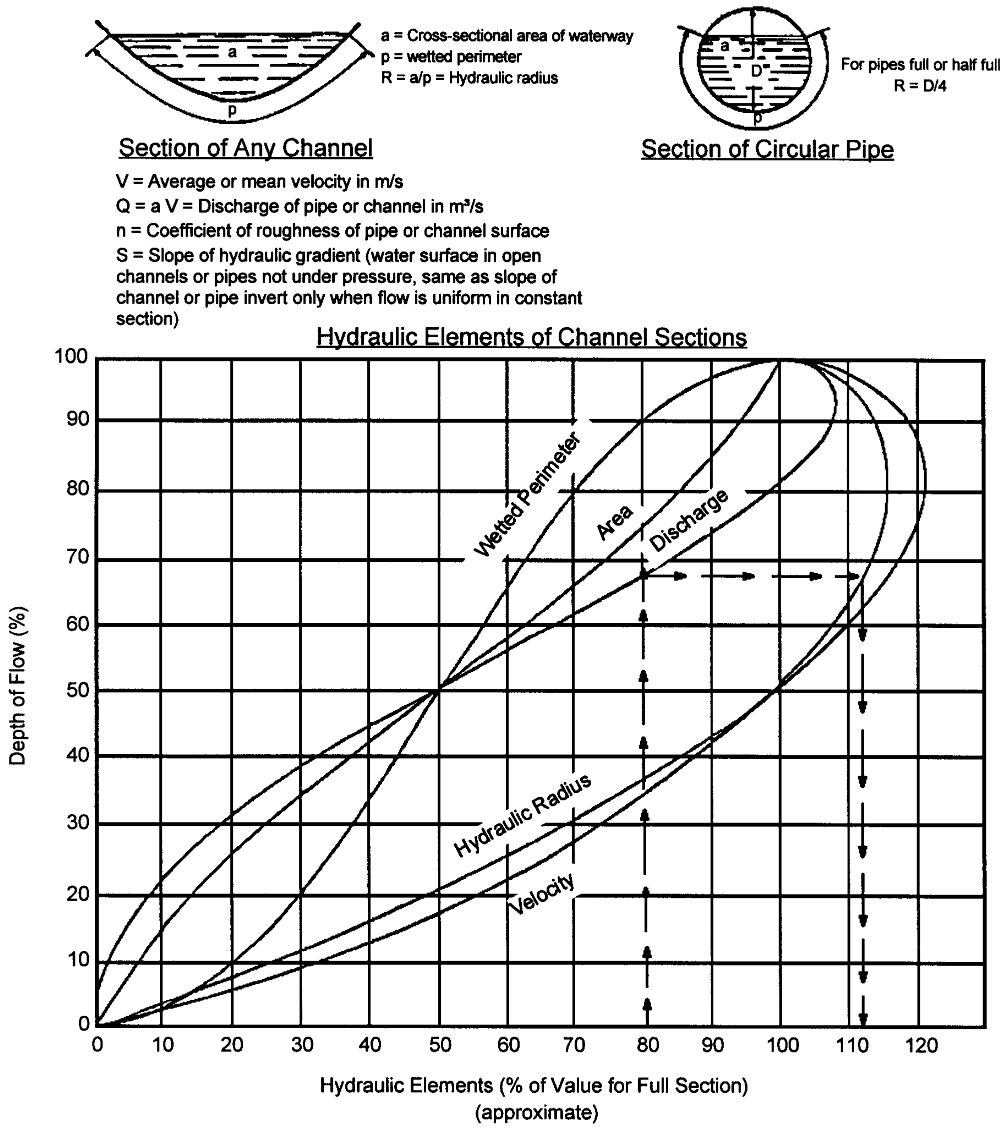


Chart of Hydraulic properties related to flow depth ratio.

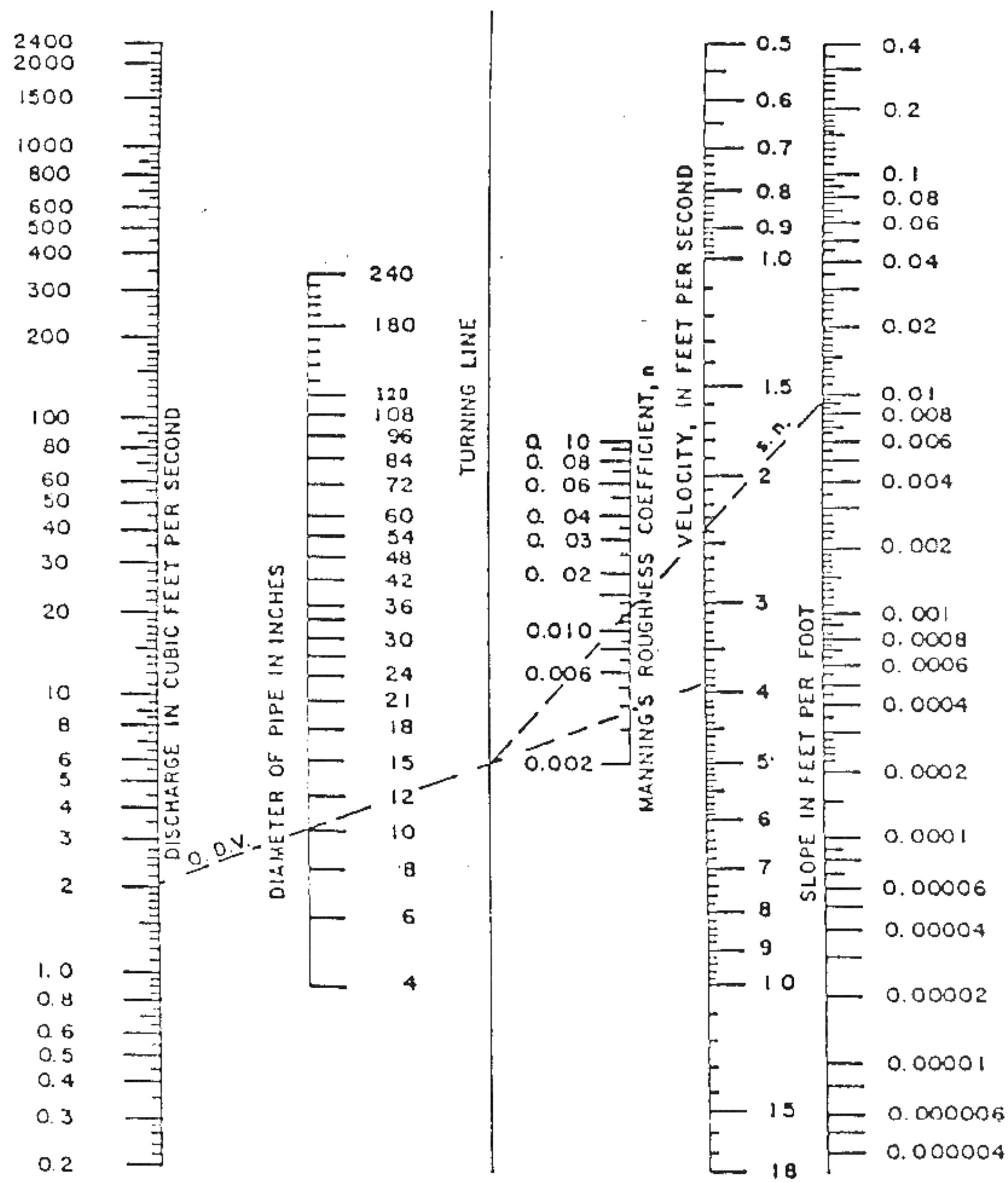
Appendix 13.B Geometric Properties of Circular Channels

TABLE 13-B.3
Pipe Reference Data
Minimum Slope is for a velocity of 3 ft/sec.

$$S = (0.004369)/D^{1.33}$$

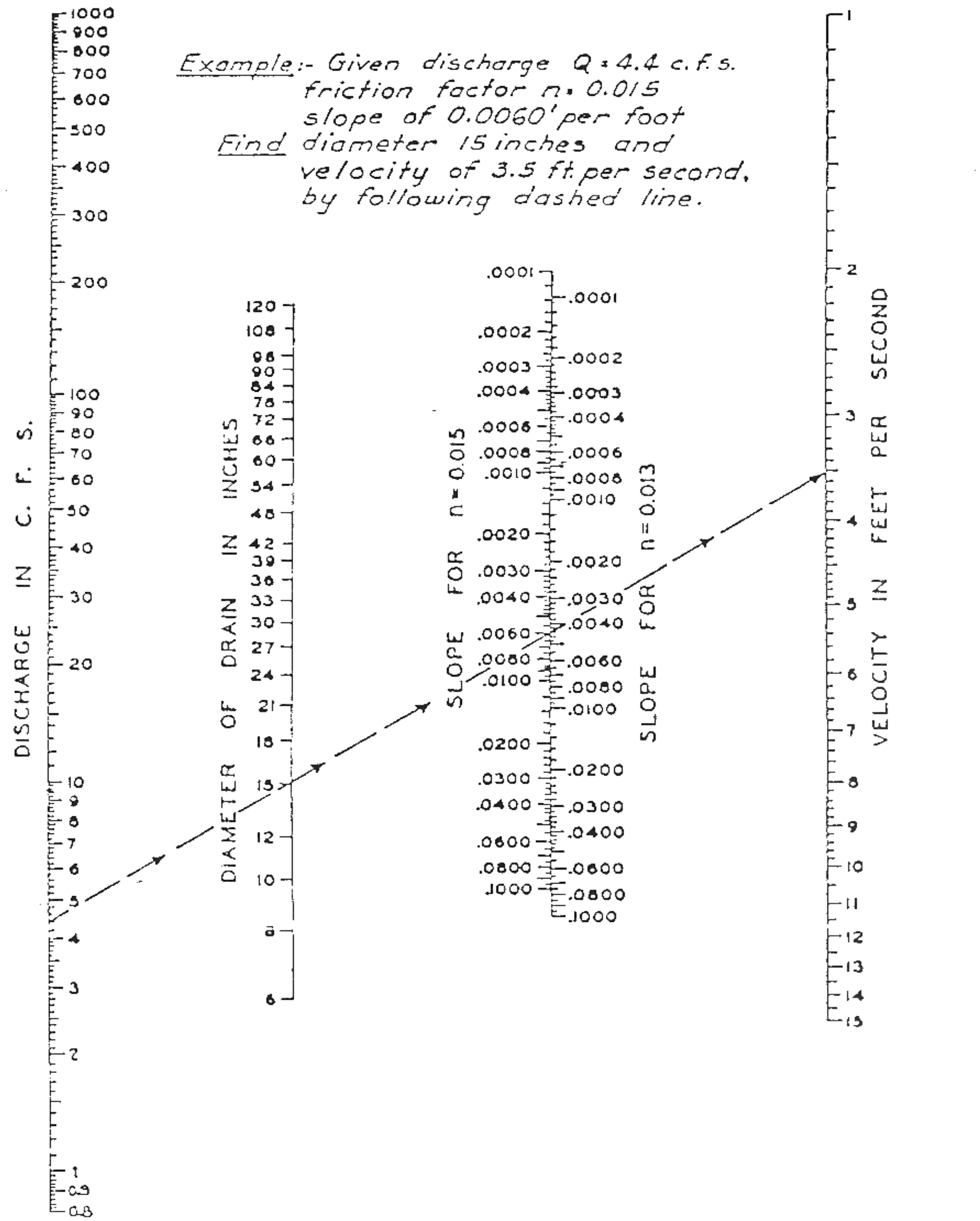
Diameter, Inch	Area, Sq. Ft.	Minimum Slope	Full Flow Discharge, cfs
18	1.767	0.0026	5.30
24	3.142	0.0017	9.43
30	4.909	0.0013	14.73
36	7.069	0.0010	21.21
42	9.621	0.00082	28.86
48	12.566	0.00069	37.70
54	15.904	0.00059	47.71
60	19.635	0.00051	58.90
66	23.758	0.00045	71.27
72	28.274	0.00040	84.22
84	38.484	0.00033	115.4
96	50.266	0.00027	150.8

Appendix 13.C - Storm Drain Design Data and Flow Charts



Flow in Storm Drain

Appendix 13.C - Storm Drain Design Data and Flow Charts



Pipe Flow Nomograph

Appendix 13.C - Storm Drain Design Data and Flow Charts

K for single pipe connection at an access structure.

For a single pipe connection, i.e. one pipe in and one pipe out,

$$K = K_o C_D C_d C_Q C_p C_B \qquad \textbf{(13.19)}$$

For a full flow pipe the above values are multiplied

C_D = correction factor for pipe diameter (pressure flow only) =1
C_d = correction factor for flow depth (non-pressure flow only) =0.5
C_Q = correction factor for relative flow= 1
C_B = correction factor for benching =
C_p = correction factor for plunging flow = 1

$$K_o = 0.1(b/ D_o)(1\text{-}\sin(\square)) + 1.4(b/ D_o)^{0.15} \sin(\square) \qquad \textbf{(13.20)}$$

Therefore K= 0.5 K_o

For b=48 inches

$$K_o = 0.1(48/ D_o)(1\text{-}\sin(\square)) + 1.4(48/ D_o)^{0.15} \sin(\square) \qquad \textbf{(13.20)}$$

Table 13-C.1
K for pipe in a manhole.

D	K_o =	□ = 180□ sin (□) = 0.0	□ = 157.5□ sin (□) = 0.383	□ = 135□ sin (□)=0.707	□ = 90□ sin (□)= 1.0
24	0.20(1-sin (□)) +1.55 sin (□)	0.1	0.36	0.575	0.775
30	0.16(1-sin (□)) +1.50 sin (□)	0.08	0.335	0.555	0.75
36	0.13(1-sin (□)) +1.46 sin (□)	0.065	0.32	0.535	0.73
42	0.11(1-sin (□)) +1.43 sin (□)	0.055	0.31	0.52	0.715
48	0.10(1-sin (□)) +1.40 sin (□)	0.05	0.30	0.51	0.70
54	0.088(1-sin (□)) +1.375 sin (□)	0.045	0.29	0.50	0.69
60	0.080(1-sin (□)) +1.354 sin (□)	0.04	0.285	0.49	0.675

Appendix 13.D - Storm Drain Design Forms

Storm Drain Computation Form

Storm Drain Computation Sheet

Computed ________ Date ________

Checked ________ Date ________

Route ________

Section ________

County ________

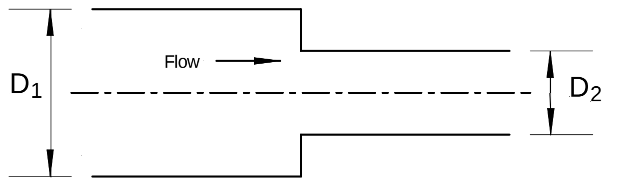
Station		Length (m)	Drainage Area A (ha)		Runoff Coefficient C	A x C		Flow Time (min)		Rainfall Intensity I (mm/h)	Total Runoff 0.0278 CIA = Q (m^3/s)	Diameter Pipe (mm)	Capacity Full (m^3/s)	Velocity (m/s)		Inert Elev.		Manhole Invert Drop	Slope of Drain (m/m)
From	To		Increment	Total		Increment	Total	To Upper End	In Section					Flowing Full	Design Flow	Upper End	Lower End		

Appendix 13.D - Storm Drain Design Forms

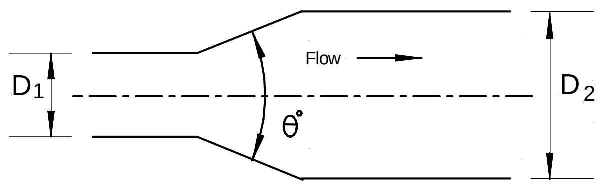
Hydraulic Grade Line Computation Form.

Station	TW HGL_o	D_0	Q_o	L_o	V_o	$V_o^2/2g$	H_o	S_{fo}	H_f	K_o	C_D	C_d	C_Q	C_p	C_B	K	$K(V_o^2/2g)$	EGL_0 2+7	EGL_i 10+18 +19	HGL_i EGL_i-7	Crown Elev
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)

Appendix E Transition Losses (Expansion and Contraction)



Sudden Contraction



Gradual Expansion

Energy Loss Coefficients
Expansion or Contraction
Figure E-1 Contraction and Expansion ,
Sudden and Gradual

Open Channel Flow

Expansion

$$h = \frac{k_e(V_1^2 - V_2^2)}{2g}$$

Contraction

$$h = \frac{k_c*(V_2^2 - V_1^2)}{2g}$$

Where

h = headloss due to expansion or contraction, ft.
k = coefficient for expansion or contraction
V_1= velocity at upstream end.
V_2= velocity at downstream end.
g = acceleration due to gravity, 32.2 ft/sec^2

Pressure Flow

$$h = \frac{k*(V_2^2)}{2g}$$

Where

h = headloss due to expansion or contraction, ft.
k = coefficient for expansion or contraction
V= velocity of flow in the smaller diameter pipe.
g = acceleration due to gravity, 32.2 ft/sec^2

Appendix E Transition Losses (Expansion and Contraction) (continued)

Table E-1
Energy Loss Coefficient, k_e
Open Channel Flow
Gradual Expansion

D_2/D_1	Angle of Cone, Degrees						
	10	**20**	**45**	**60**	**90**	**120**	**180**
1.5	0.17	0.40	1.06	1.21	1.14	1.07	1.00
3.0	0.17	0.40	0.86	1.02	1.06	1.04	1.00
D_2/D_1 = Ratio of diameter of large pipe to diameter of smaller pipe. Angle of cone in degrees between the sides of the tapering section.							

For gradual contractions $k_c = 0.5*k_e$.

Table E-2
Energy Loss Coefficient, k_c
Open Channel Flow
Sudden Contraction

D_2/D_1	k_c
0	0.5
0.4	0.4
0.6	0.3
0.8	0.1
1.0	0.0
D_2/D_1 = Ratio of diameter of large pipe to diameter of smaller pipe.	

Appendix E Transition Losses (Expansion and Contraction) (continued)

Table E-3
Coefficient k_e
Pressure Flow

D_2/D_1	Angle of Cone, degrees													
	2	**4**	**6**	**8**	**10**	**15**	**20**	**25**	**30**	**35**	**40**	**45**	**50**	**60**
1.1	0.01	0.01	0.01	0.02	0.03	0.05	0.10	0.13	0.16	0.18	0.19	0.20	0.21	0.23
1.2	0.02	0.02	0.02	0.03	0.04	0.09	0.16	0.21	0.25	0.29	0.31	0.33	0.35	0.37
1.4	0.02	0.03	0.03	0.04	0.06	0.12	0.23	0.30	0.36	0.41	0.44	0.47	0.50	0.53
1.6	0.03	0.03	0.04	0.05	0.07	0.14	0.26	0.35	0.42	0.47	0.51	0.54	0.57	0.61
1.8	0.03	0.04	0.04	0.05	0.07	0.15	0.28	0.37	0.44	0.50	0.54	0.58	0.61	0.65
2.0	0.03	0.04	0.04	0.05	0.08	0.16	0.30	0.39	0.48	0.54	0.58	0.62	0.65	0.70
2.5	0.03	0.04	0.04	0.05	0.08	0.16	0.30	0.39	0.48	0.55	0.59	0.63	0.66	0.71
3.0	0.03	0.04	0.04	0.05	0.08	0.16	0.31	0.40	0.48	0.55	0.59	0.63	0.66	0.71
	0.03	0.04	0.05	0.06	0.08	0.16	0.31	0.40	0.49	0.56	0.60	0.64	0.67	0.72

D_2/D_1 = Ratio of diameter of large pipe to diameter of smaller pipe.
Angle of cone in degrees between the sides of the tapering section.

Gradual Enlargement

Table E-4
Coefficient k_e
Pressure Flow
Sudden Enlargement

D_2/D_1	Velocity, V_1, ft/sec												
	2	**3**	**4**	**5**	**6**	**7**	**8**	**10**	**12**	**15**	**20**	**30**	**40**
1.2	0.11	0.10	0.10	0.10	0.10	0.10	0.10	0.09	0.09	0.09	0.09	0.09	0.08
1.4	0.26	0.26	0.25	0.24	0.24	0.24	0.24	0.23	0.23	0.22	0.22	0.21	0.20
1.6	0.40	0.39	0.38	0.37	0.37	0.36	0.36	0.35	0.35	0.34	0.33	0.32	0.32
1.8	0.51	0.49	0.48	0.47	0.47	0.46	0.46	0.45	0.44	0.43	0.42	0.41	0.40
2.0	0.60	0.58	0.56	0.55	0.55	0.54	0.53	0.52	0.52	0.51	0.50	0.48	0.47
2.5	0.74	0.72	0.70	0.69	0.68	0.67	0.66	0.65	0.64	0.63	0.62	0.60	0.58
2.5	0.74	0.72	0.70	0.69	0.68	0.67	0.66	0.65	0.64	0.63	0.62	0.60	0.58
3.0	0.83	0.80	0.78	0.77	0.76	0.75	0.74	0.73	0.72	0.70	0.69	0.67	0.65
4.0	0.92	0.89	0.87	0.85	0.84	0.83	0.82	0.80	0.79	0.78	0.76	0.74	0.72
5.0	0.96	0.93	0.91	0.89	0.88	0.87	0.86	0.84	0.83	0.82	0.80	0.77	0.75
10.0	1.00	0.99	0.96	0.95	0.93	0.92	0.91	0.89	0.88	0.86	0.84	0.82	0.80
	1.00	1.00	0.98	0.96	0.95	0.94	0.93	0.91	0.90	0.88	0.86	0.83	0.81

D_2/D_1 = Ratio of diameter of large pipe to diameter of smaller pipe.
V_1 = Velocity in smaller pipe.

Appendix E Transition Losses (Expansion and Contraction) (continued)

Table E-5
Coefficient k_c
Pressure Flow
Sudden Contraction

D_1/D_2	Velocity, V_1, ft/sec												
	2	**3**	**4**	**5**	**6**	**7**	**8**	**10**	**12**	**15**	**20**	**30**	**40**
1.1	0.03	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.04	0.05	0.05	0.06
1.2	0.07	0.07	0.07	0.07	0.07	0.07	0.07	0.08	0.08	0.08	0.09	0.10	0.11
1.4	0.17	0.17	0.17	0.17	0.17	0.17	0.17	0.18	0.18	0.18	0.18	0.19	0.50
1.6	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.26	0.25	0.25	0.25	0.24
1.8	0.34	0.34	0.34	0.34	0.34	0.34	0.33	0.33	0.32	0.32	0.31	0.29	0.27
2.0	0.38	0.8	0.37	0.37	0.37	0.37	0.36	0.36	0.35	0.34	0.33	0.31	0.29
2.2	0.40	0.40	0.40	0.39	0.39	0.39	0.39	0.38	0.37	0.37	0.35	0.33	0.30
2.5	0.42	0.42	0.42	0.41	0.41	0.41	0.40	0.40	0.39	0.38	0.37	0.34	031
3.0	0.44	0.44	0.44	0.43	0.43	0.43	0.42	0.42	0.41	0.40	0.39	0.36	0.33
4.0	0.47	0.46	0.46	0.46	0.45	0.45	0.45	0.44	0.43	0.42	0.41	0.37	0.34
5.0	0.48	0.48	0.47	0.47	0.47	0.46	0.46	0.45	0.45	0.44	0.42	0.38	0.35
10.0	0.49	0.48	0.48	0.48	0.48	0.47	0.47	0.46	0.46	0.45	0.43	0.40	0.36
	0.49	0.49	0.48	0.48	0.48	0.47	0.47	0.47	0.46	0.45	0.44	0.41	0.38

D_2/D_1 = Ratio of diameter of large pipe to diameter of smaller pipe.
V_1 = Velocity in smaller pipe.